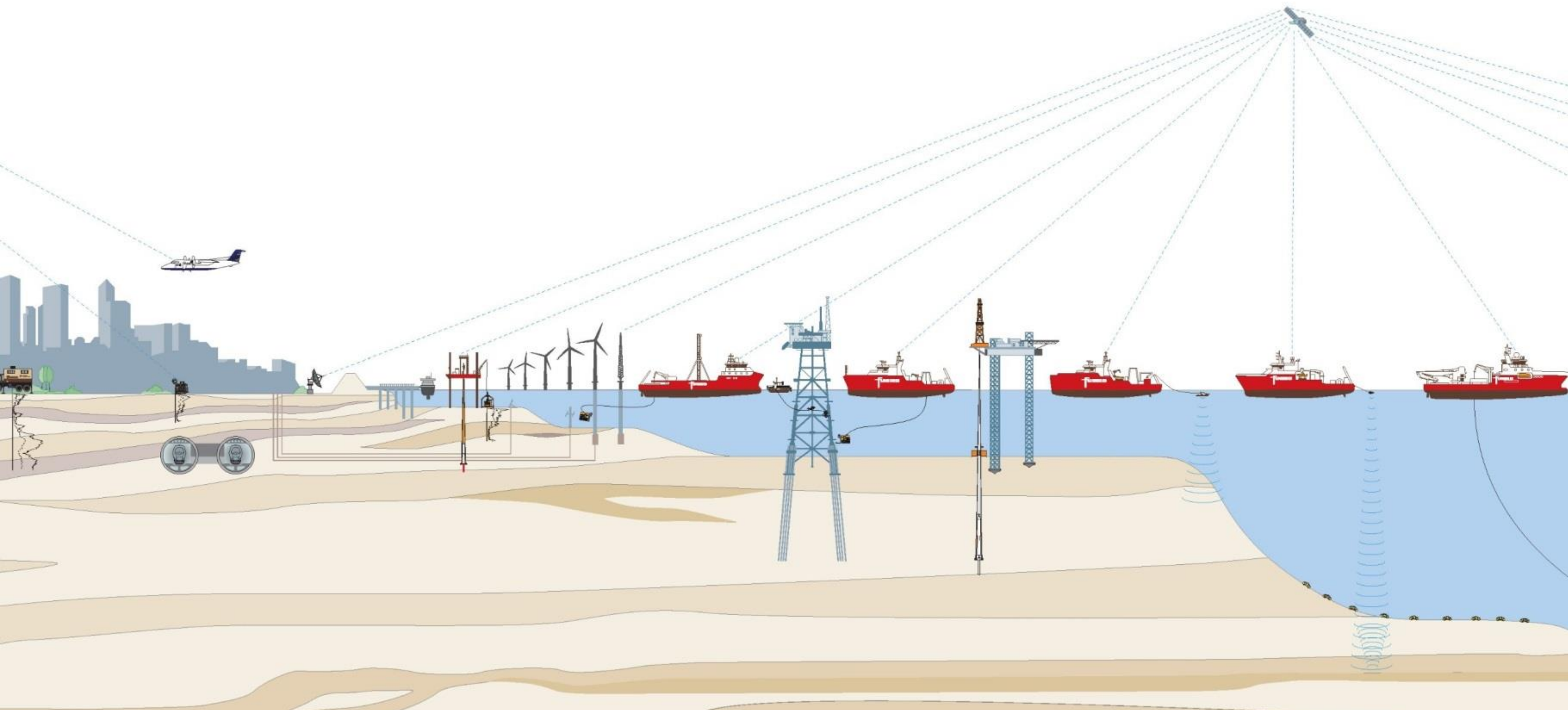
Geotechnical Report
Investigation Data
Sillimanite Pipeline Routes
Dutch Sector, North Sea

Fugro Report No. GH210-R3
Issue 1

Wintershall Noordzee B.V.



Geotechnical Report
Investigation Data
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Issue 1

Client	Wintershall Noordzee B.V.
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Fugro Project Lead J. Lakeman - Senior Project Engineer

1	Final	17 May 2017
0	Provisional on-site data	Various
Issue	**Document Status**	**Date**

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ISSUE 85
FEBV/GEO/REP/001
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1. OVERVIEW

1.1 Purpose and Scope of Report

Wintershall Noordzee BV, henceforth referred to as Wintershall (Client) is planning the development of a pipeline between Sillimanite and D15 and between Sillimanite and D12, henceforth referred to as "Sillimanite Pipeline Routes", refer also to Plates 1 and 2. The routes are located in the Dutch Sector of the North Sea, in water depths ranging from approximately 29 m to 38 m relative to Lowest Astronomical Tide (LAT).

A project-specific investigation was commissioned to acquire geophysical and geotechnical information along the Sillimanite Pipeline Routes in support of the planned development. This report presents results of the geotechnical investigation. Results of the geophysical survey are presented in a companion project report. The geotechnical investigation included seafloor in situ testing, i.e. cone penetration testing, only. Location details for the various test points are provided on Plates 3 and 4.

Information presented in this report comprises the following:

- Geotechnical logs for nineteen locations to depth from approximately 1.8 m to 3.3 m below seafloor (BSF);
- Results of seafloor cone penetration testing at nineteen test points.

1.2 Project Responsibilities and Use of Report

This report presents information according to a project specification determined and monitored by the Client. The Client approved the investigation programme. During the investigation, the Client's programme was adjusted to suit as-found conditions and operational constraints. The scope of the report includes the results of the final programme. The Main Text section titled "Sources of Information and References" provides further details.

This report must be read in conjunction with the section titled "Use of Report". This section includes information about report issue control.

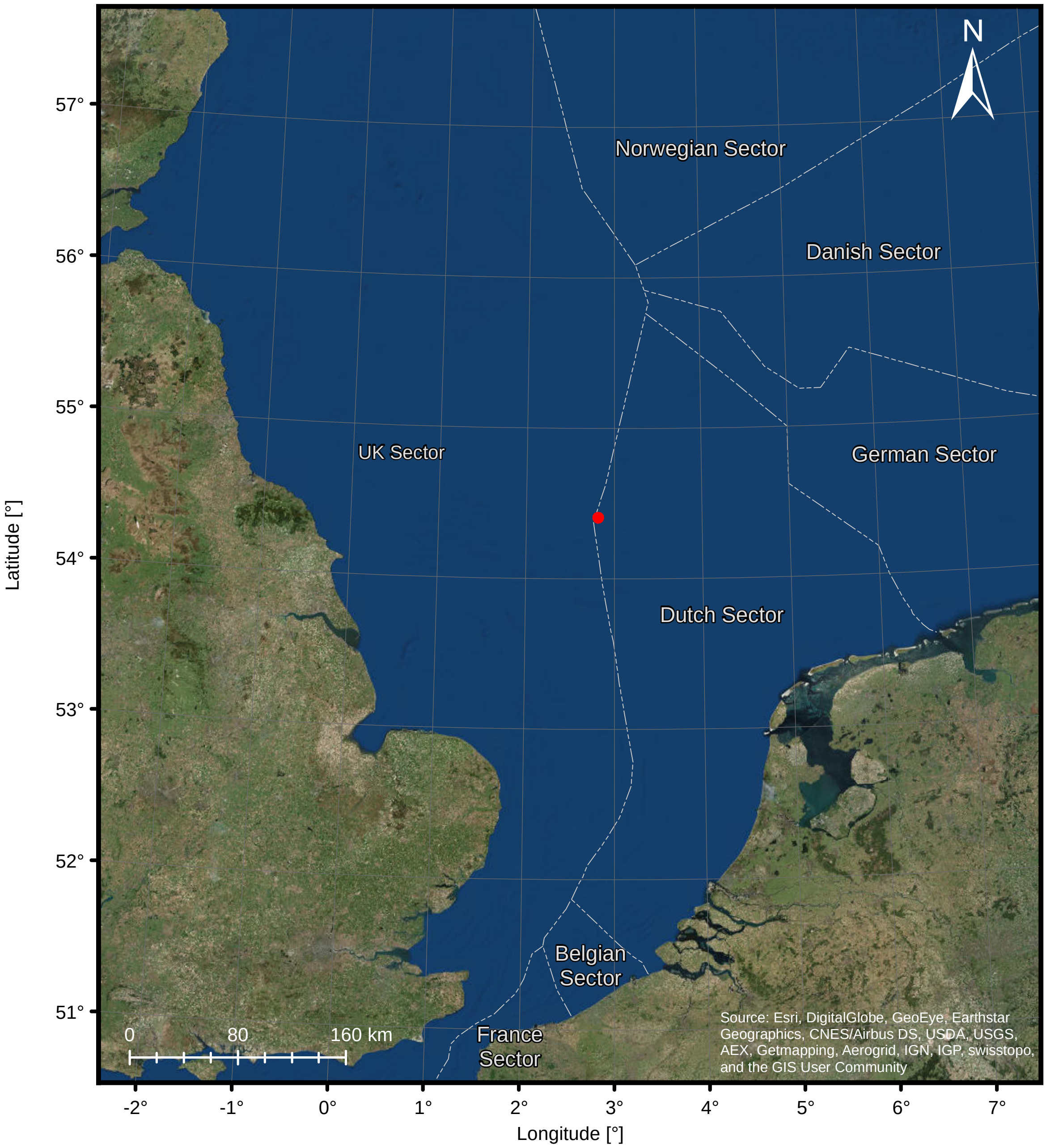
Fugro understands that this report will be used for the purpose described in this Main Text section. That purpose was a significant factor in determining the scope and level of the services. Results must not be used if the purpose for which the report was prepared or the Client's proposed development or activity changes. Results may possibly suit alternative use. Suitability must be verified.

2. SOURCES OF INFORMATION AND REFERENCES

2.1 Fugro Information

This report uses and summarises Fugro-held information:

- Fugro data base:
  - Information about regional geology;
  - General geotechnical data;
  - Previous geotechnical investigation data applicable to nearby sites.



Maritime boundary

Approximate area of investigation

Datum	:	European Datum 1950 (ED50)
Spheriod	:	International 1924
Projection	:	Universal Transverse Mercator (UTM)
Central Meridian	:	3° East (UTM Zone 31 N)

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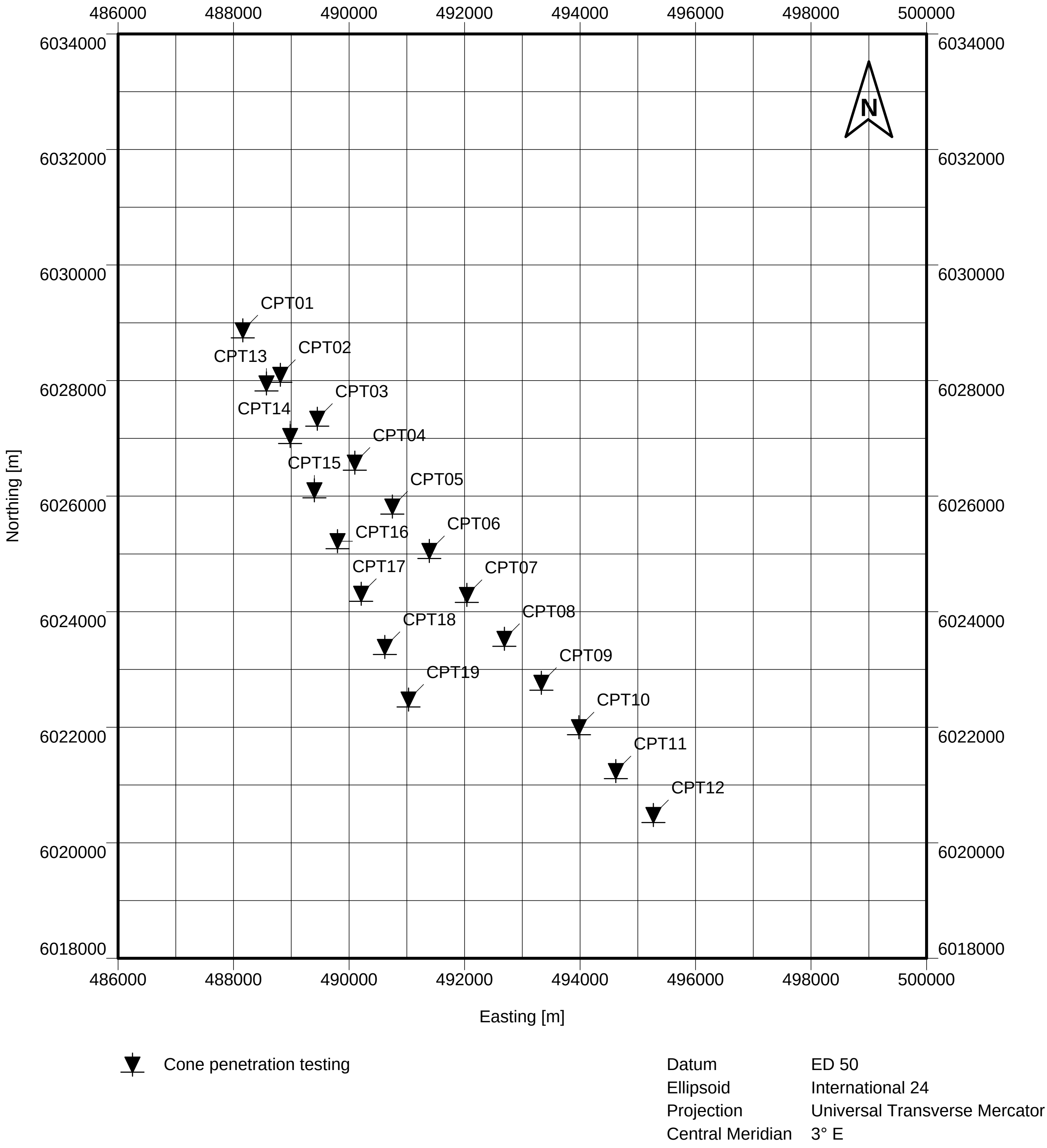
VICINITY MAP



GeODin/P902940 - Detailed Location Plan.GLO/2017-05-17 10:32

DETAILED LOCATION PLAN

COORDINATES

Location	Penetration Depth [m]	Easting [m]	Northing [m]	Latitude [deg]	Longitude [deg]
CPT01	1,8	488162	6028866	54°24'22,4" N	2°49'3,5" E
CPT02	3,3	488807	6028105	54°23'57,8" N	2°49'39,3" E
CPT03	3,3	489454	6027342	54°23'33,2" N	2°50'15,3" E
CPT04	3,3	490101	6026576	54°23'8,5" N	2°50'51,3" E
CPT05	3,3	490747	6025815	54°22'43,9" N	2°51'27,2" E
CPT06	3,3	491394	6025052	54°22'19,3" N	2°52'3,1" E
CPT07	3,3	492040	6024287	54°21'54,6" N	2°52'39,0" E
CPT08	3,3	492687	6023526	54°21'30,0" N	2°53'14,9" E
CPT09	3,3	493327	6022770	54°21'5,5" N	2°53'50,4" E
CPT10	3,3	493979	6021999	54°20'40,6" N	2°54'26,6" E

Datum : ED 50 Projection : Universal Transverse Mercator
Ellipsoid : International 24 Central Meridian : 3° E
Penetration Depth: deepest point reached by drilling, sampling or in situ testing, relative to seafloor
For further details, refer to plate titled “Geodetic Parameters”

WATER DEPTH

Location	Date	Bathymetry Chart Reduced [m]
CPT01	19-apr-2017	28,6
CPT02	19-apr-2017	28,9
CPT03	19-apr-2017	29,3
CPT04	19-apr-2017	29,6
CPT05	19-apr-2017	30,0
CPT06	19-apr-2017	30,6
CPT07	19-apr-2017	31,2
CPT08	19-apr-2017	31,9
CPT09	19-apr-2017	33,1
CPT10	19-apr-2017	34,6

Reduced water depths are relative to Lowest Astronomical Tide (LAT)

GeODin/GH210-R2 - Coordinates and Water Depth.GLO/2017-05-17 10:31

COORDINATES AND WATER DEPTH

COORDINATES

Location	Penetration Depth [m]	Easting [m]	Northing [m]	Latitude [deg]	Longitude [deg]
CPT11	3,3	494625	6021237	54°20'16,0" N	2°55'2,4" E
CPT12	3,3	495273	6020475	54°19'51,4" N	2°55'38,3" E
CPT13	3,3	488571	6027954	54°23'53,0" N	2°49'26,3" E
CPT14	3,3	488980	6027041	54°23'23,4" N	2°49'49,1" E
CPT15	3,3	489403	6026101	54°22'53,1" N	2°50'12,7" E
CPT16	3,3	489801	6025217	54°22'24,5" N	2°50'34,8" E
CPT17	3,3	490211	6024306	54°21'55,1" N	2°50'57,7" E
CPT18	3,3	490619	6023393	54°21'25,5" N	2°51'20,4" E
CPT19	3,3	491029	6022481	54°20'56,1" N	2°51'43,1" E

Datum : ED 50 Projection : Universal Transverse Mercator
Ellipsoid : International 24 Central Meridian : 3° E
Penetration Depth: deepest point reached by drilling, sampling or in situ testing, relative to seafloor
For further details, refer to plate titled “Geodetic Parameters”

WATER DEPTH

Location	Date	Bathymetry Chart Reduced [m]
CPT11	19-apr-2017	36,3
CPT12	20-apr-2017	38,2
CPT13	19-apr-2017	28,8
CPT14	19-apr-2017	29,1
CPT15	20-apr-2017	29,4
CPT16	20-apr-2017	29,8
CPT17	20-apr-2017	30,3
CPT18	20-apr-2017	30,8
CPT19	20-apr-2017	31,4

Reduced water depths are relative to Lowest Astronomical Tide (LAT)

GeODin/GH210-R2 - Coordinates and Water Depth.GLO/2017-05-17 10:31

COORDINATES AND WATER DEPTH

DGPS Geodetic Parameters		
Datum		WGS84 (World Geodetic System 1984)
Spheroid		WGS84 (World Geodetic System 1984)
Semi-Major Axis, a		6378137.000 m
Inverse Flattening, 1/f		298.2572236
Transformation Parameters (from WGS84 to Local Grid)		
Source Shift		
dX		+89.5 m
dY		+93.8 m
dZ		+123.1 m
Rotation and Scale		
rX		0.0"
rY		0.0"
rZ		0.2"
dS (Scale Factor)		-1.2 ppm
Local Grid Geodetic Parameters		
Datum		ED50 (European Datum 1950)
Spheroid		International 1924
Semi-Major Axis, a		6378388.000 m
Inverse Flattening, 1/f		297
Local Projection Parameters		
Projection		UTM (Universal Transverse Mercator)
Hemisphere		Northern
Central Meridian (CM)		03° 00' 00.0000" E
Latitude of Origin		00° 00' 00.0000" N
False Easting		500000 m
False Northing		0 m
Scale Factor on CM		0.9996
Units		metres
Example Coordinates		
Local grid coordinates	Easting	500000.0 m
	Northing	5649824.9 m
Local geographical coordinates	Latitude	51° 00' 00.0000" N
	Longitude	03° 00' 00.0000" E
WGS84 geographical coordinates	Latitude	51° 00' 00.0000" N
	Longitude	03° 00' 00.0000" E

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GEODETIC PARAMETERS

Overview

General Procedure:	– According to ISO (2014) – Metrological confirmation according to ISO (2003) – Refer to document titled “Positioning Survey and Depth Measurement” presented in Appendix 1
Purpose of Measurement(s):	– Provision of spatial (xyz) position of data points applicable to geotechnical investigation location(s) – The user of the presented positioning survey and depth measurement data must consider the purpose and accuracy of measurements, particularly where use may differ from original intentions
Geodetic Parameters:	Refer to plate(s) titled "Geodetic Parameters"
Vertical Datum:	– For water depth: Lowest Astronomical Tide LAT – For geotechnical investigation data: depth below seafloor

Data Acquisition System(s)

Primary System for Positioning Survey:	GNSS positioning of vessel antennae (2 independent systems)
Primary System for Water Depth Measurement:	Multi beam echosounder, vessel hull mounted
Primary System for Investigation Data Points below Vertical Datum:	Refer to report section(s) on investigation data for measurement of depth below seafloor, e.g. by depth (distance) transducer, inclinometer etc.
Secondary System(s):	Subsurface positioning by ultra short baseline USBL system on seabed frame relative to vessel;

Results

Data Processing:	Fugro in-house software
Data Format(s):	PDF for viewing and printing (this primary document)
Conversion of Coordinates Systems:	Refer to plate(s) titled "Geodetic Parameters"
Correlation of Depth Measurements:	Not considered, overall inspection of multiple data sets
Depth Correction(s) for Sloping or Irregular Seafloor:	Not considered
Depth Correction(s) for Very Soft Seabed:	Not applicable
Positioning Survey and Depth Measurement Data:	– Refer to plate(s) titled "Coordinates and Water Depth" – Refer to report section(s) on investigation data for elevation (depth) of data points below vertical datum – Presented water depth measurements serve to establish logging, testing and/or sampling depths below seafloor only

References

– International Organization for Standardization, 2003. *ISO 10012:2003 Measurement Management Systems - Requirements for Measurement Processes and Measuring Equipment*. Geneva: ISO.

– International Organization for Standardization, 2014. *ISO 19901-8:2014 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures – Part 8: Marine Soil Investigations.* Geneva: ISO.

PRACTICE FOR POSITIONING SURVEY AND DEPTH MEASUREMENT

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SECTION A: GEOTECHNICAL LOGS

TEXT – SECTION A: Page

LIST OF PLATES IN SECTION A: Plate

A. GEOTECHNICAL LOGS

A.1 PRACTICE FOR GEOTECHNICAL LOG

Approach

Purpose:	Information in support of development of pipeline routes
General Procedure:	– Refer to document titled "Geotechnical Log" presented in Appendix 1 – According to ISO (2014)

Results

Data Processing and Interpretation:	– UNIPLOT software – Graphical scales selected to suit general presentation of data – No display of data outside of chart limits, i.e. some values may not be shown – Geotechnical description is an interpretation of processed data available at the time of preparation; for example, interfaces between strata may be more gradual than a log indicates – Level of detail and accuracy in geotechnical description and interpretation depend on factors such as investigation system(s), logging/ test data, sample size, quality, coverage, availability of supplementary information, and project requirements
Data Format(s):	PDF for viewing and printing (this primary document)
Matching Test, Sample and Logging Data:	Not applicable, geotechnical log based on CPT data only
Ground Description:	According to classification methods proposed by Robertson (2009) and document "Cone Penetration Test Interpretation" presented in Appendix 1
Unit Weight derived from In Situ Test:	Not applicable
Relative Density derived from In Situ Test:	If applicable: – refer to document titled "Cone Penetration Test Interpretation" presented in Appendix 1 – according to Jamiolkowski et al. (2003) for saturated coarse-grained, frictional soil behaviour, Ticino Sand correlation – based on earth pressure coefficient values K_0 = 0.5 and 1.0 – relative density calculated where soil behaviour type index I_c/I_{SBT} < 2.6 – no relative density calculation for initial penetration of cone penetrometer into soil, i.e. for limited embedment of cone penetrometer – presented values represent results of correlation(s), i.e. not an expected range
Undrained Shear Strength derived from In Situ Test:	If applicable: – refer to document titled "Cone Penetration Test Interpretation" presented in Appendix 1 – applies to interpreted fine-grained, cohesive soil behaviour – based on cone factor of N_k = 15 and 20 – undrained shear strength calculated where soil behaviour type index I_c/I_{SBT} > 2.05 – no calculation of undrained shear strength for initial penetration of cone penetrometer into soil, i.e. for limited embedment of cone penetrometer
Laboratory Test Data:	Not applicable
Coordinates and Water Depth:	Applicable to test point
Water Depth Reference:	As obtained from multi beam echo sounder readings at prior to start of testing
Depth Reference Correction:	None applied

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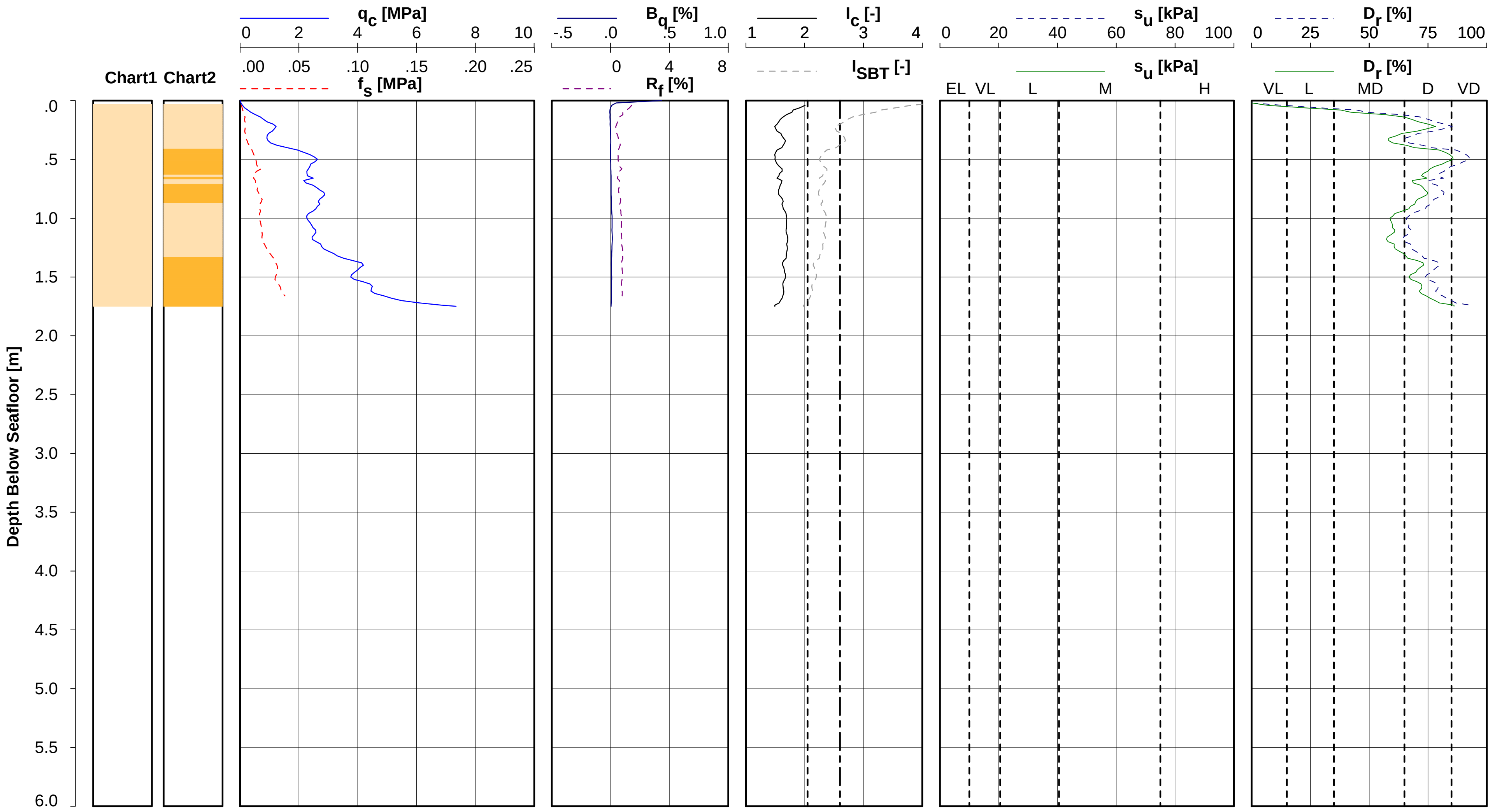
A.2 REFERENCES

- Computer Program UNIPLOT, *Processing, Presentation and Analysis of In Situ Test Data*.
- Computer Program GeODin®*, Recording, Presentation and Analysis of Geo-data.*
- British Standards Institution, 2015. *BS 5930:2015 Code of practice for ground investigations*. London: BSI.
- International Organization for Standardization, 2014. *ISO 19901-8:2014 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures – Part 8: Marine Soil Investigations.* Geneva: ISO.
- Jamiolkowski, M., Lo Presti, D.C.F. and Manassero, M. 2003. Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT. In Germaine, J.T., Sheahan, T.C. and Whitman, R.V. Eds. *Soil Behavior and Soft Ground Construction: Proceedings of the Symposium, October 5-6, 2001, Cambridge, Massachusetts,* Reston: American Society of Civil Engineers, Geotechnical Special Publication, No. 119, pp. 201-238.
- Robertson, P.K. 2009. Performance Based Earthquake Design Using the CPT. In Kokusho, T., Tsukamoto, Y. and Yoshimine, M. Eds. *Performance-Based Design in Earthquake Geotechnical Engineering – from Case History to Practice: Proceedings of the International Conference on Performance-Based Design in Earthquake Geotechnical Engineering IS-Tokyo 2009), 15-18 June 2009,* Boca Raton: CRC Press, pp. 3-20.
- Robertson, P.K. 2010. Soil Behaviour type from the CPT: an update. In *2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, Vol.2.* pp 575-583.



sensitive, fine grained
organic soils - PEATS
CLAYS - clay to silty clay
silt mixtures - clayey SILT to silty CLAY
SAND mixtures - silty sand to sandy silt
SANDS - clean sand to silty sand
gravelly sand to sand
very stiff sand to clayey sand
very stiff, fine grained

----> I_c/I_{SBT} plot s_u
—--< I_c/I_{SBT} plot D_r

EL : extremely low　L : low　H : high
VL : very low　M: medium

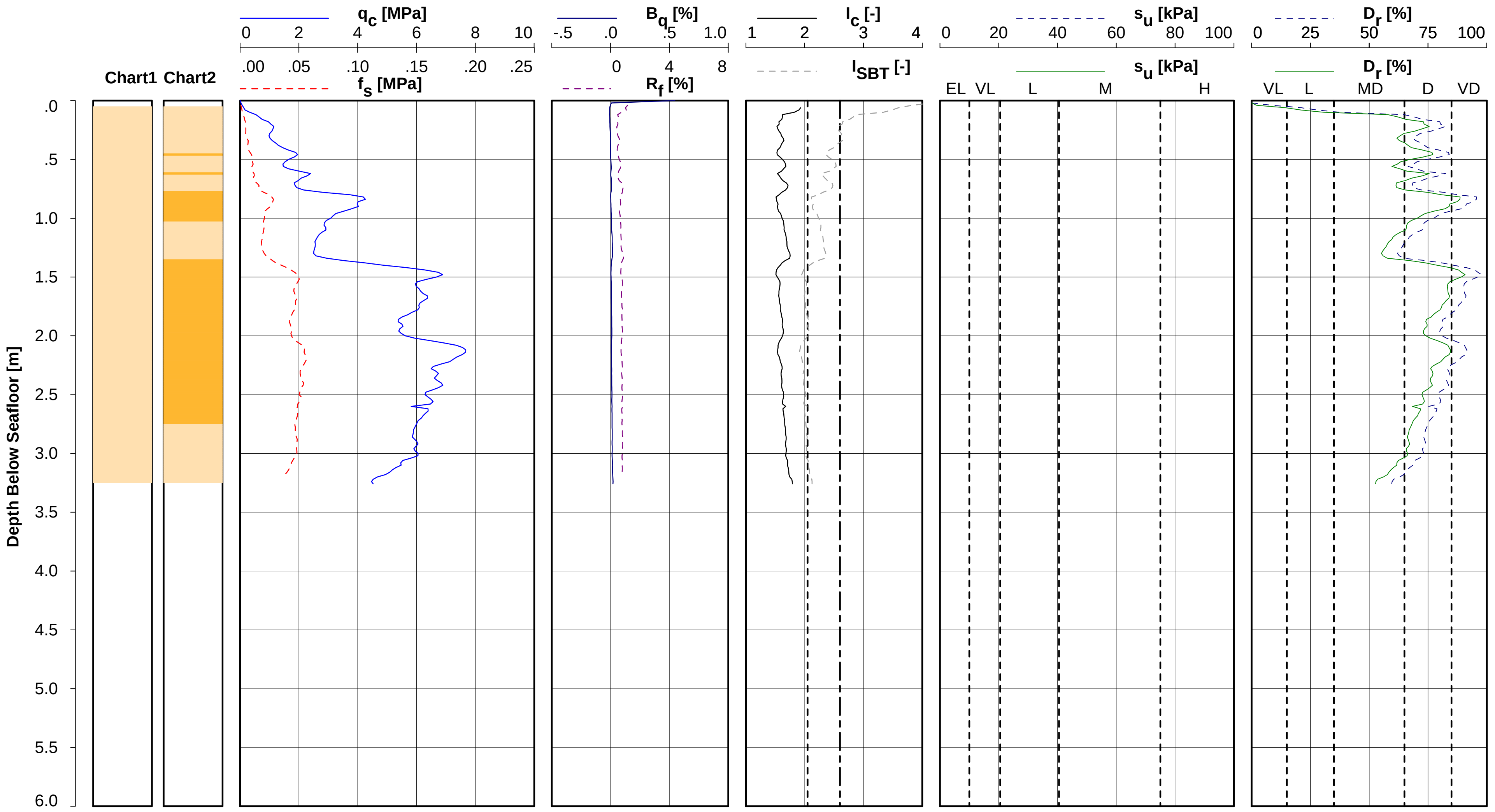
VL : very loose　D : dense
L : loose　VD: very dense
L : medium dense

Water Depth [m] : 28.6　Coordinates [m] : E 488162　N 6028866

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT01



sensitive, fine grained	SANDS - clean sand to silty sand
organic soils - PEATS	gravelly sand to sand
CLAYS - clay to silty clay	very stiff sand to clayey sand
silt mixtures - clayey SILT to silty CLAY	very stiff, fine grained
SAND mixtures - silty sand to sandy silt	

----> I_c/I_{SBT} plot s_u
—--< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

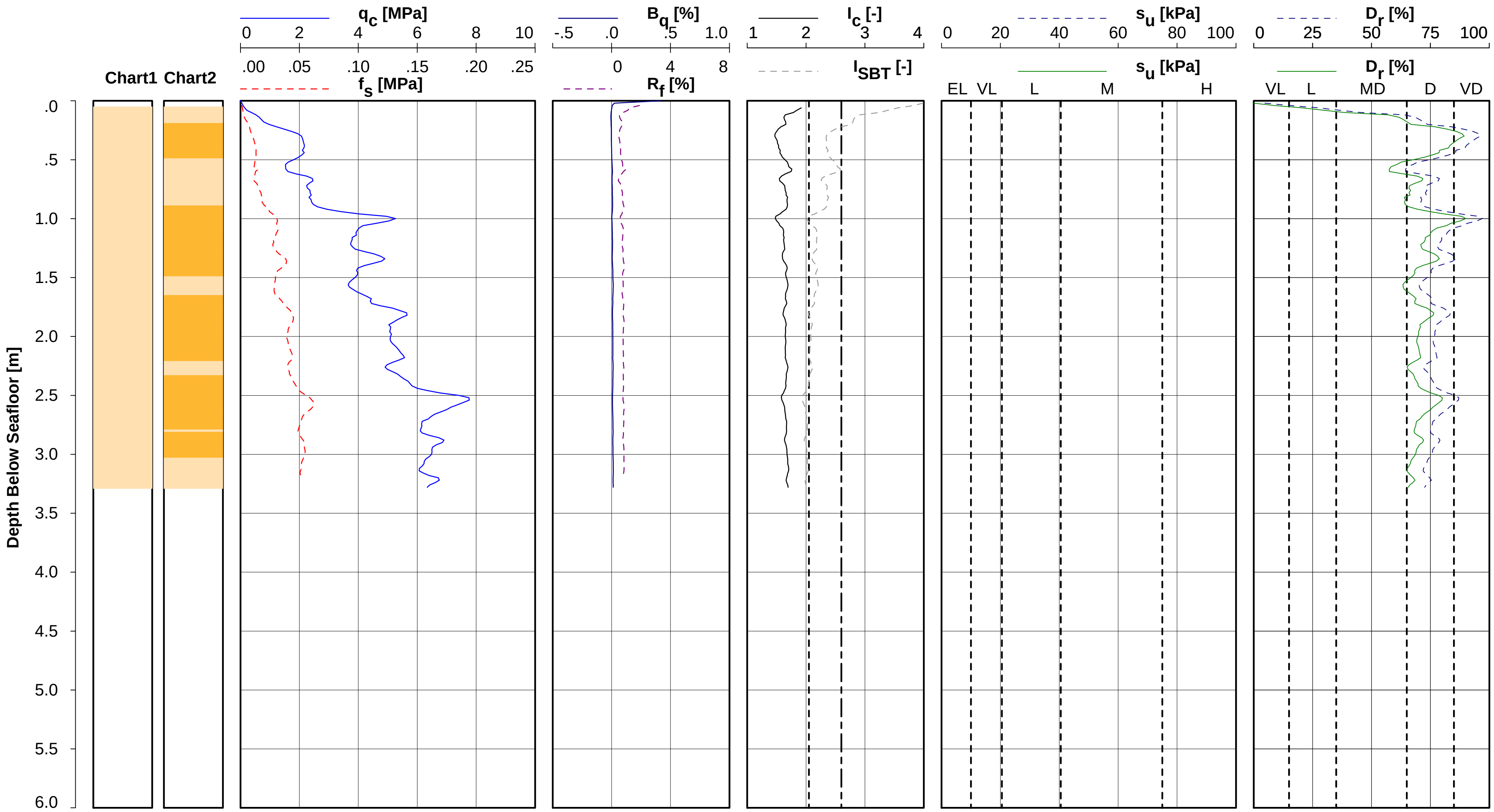
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 28.9 Coordinates [m] : E 488807 N 6028105

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT02



sensitive, fine grained

organic soils - PEATS

CLAYS - clay to silty clay

silt mixtures - clayey SILT to silty CLAY

SAND mixtures - silty sand to sandy silt

SANDS - clean sand to silty sand

gravelly sand to sand

very stiff sand to clayey sand

very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u
—— - - -< I_c/I_{SBT} plot D_r

EL : extremely low
VL : very low
L : low
M: medium
H : high

VL : very loose
L : loose
L : medium dense
D : dense
VD: very dense

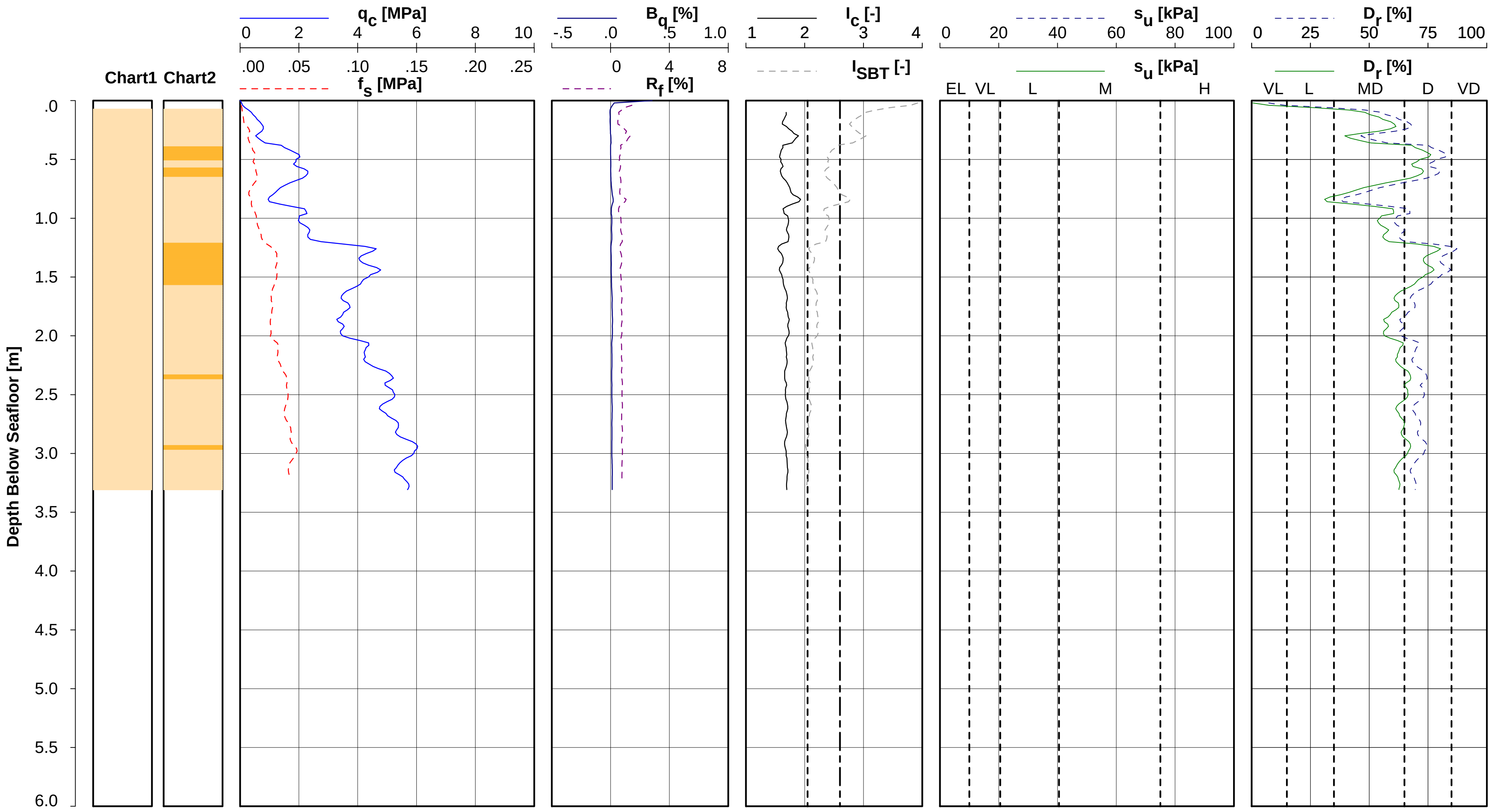
Water Depth [m] : 29.3

Coordinates [m] : E 489454 N 6027342

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT03



sensitive, fine grained
organic soils - PEATS
CLAYS - clay to silty clay
silt mixtures - clayey SILT to silty CLAY
SAND mixtures - silty sand to sandy silt
SANDS - clean sand to silty sand
gravelly sand to sand
very stiff sand to clayey sand
very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u
—— - - -< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

VL : very loose D : dense
L : loose VD: very dense
L : medium dense

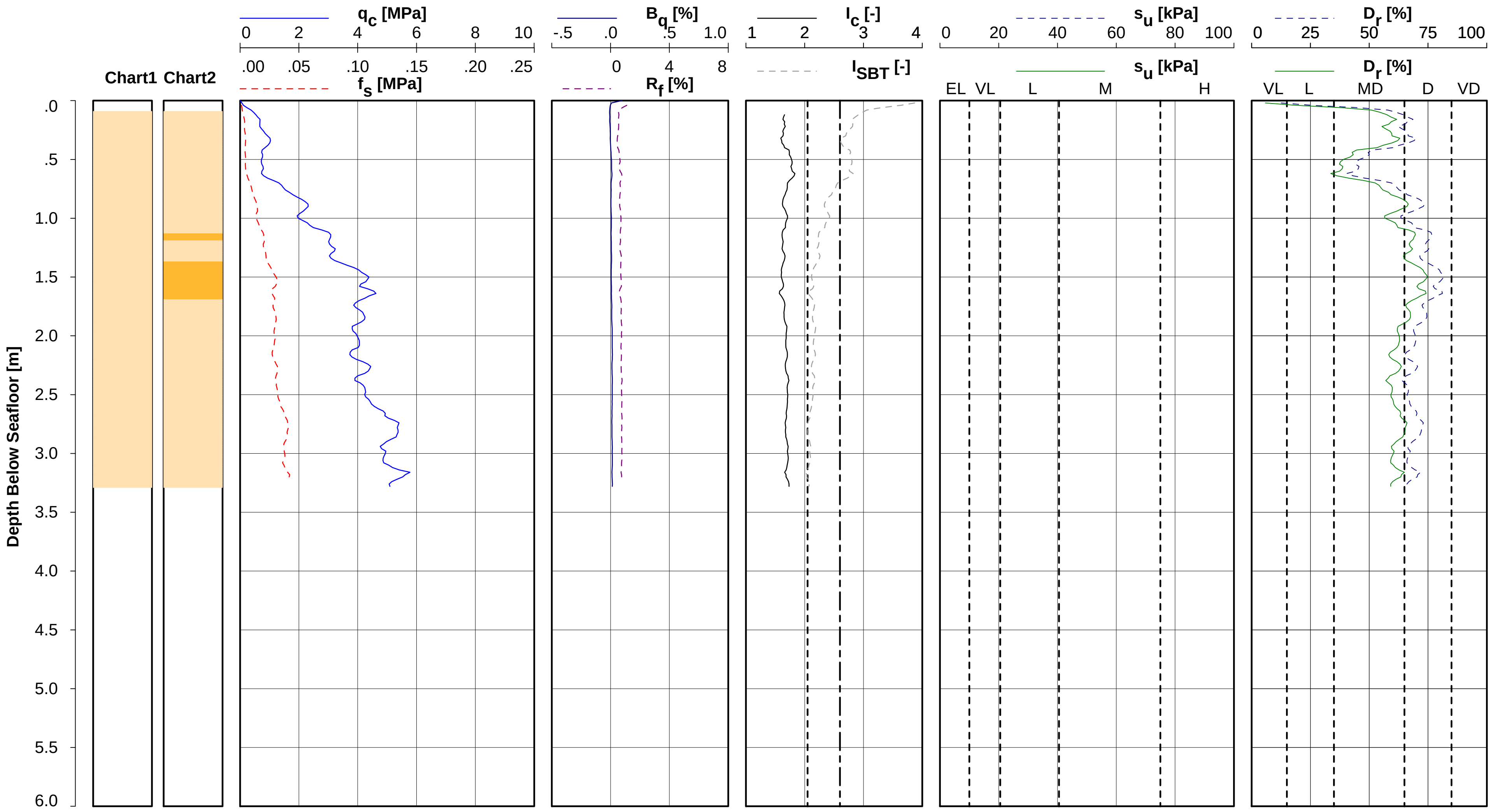
Water Depth [m] : 29.6

Coordinates [m] : E 490101 N 6026576

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT04



- sensitive, fine grained
- organic soils - PEATS
- CLAYS - clay to silty clay
- silt mixtures - clayey SILT to silty CLAY
- SAND mixtures - silty sand to sandy silt
- SANDS - clean sand to silty sand
- gravelly sand to sand
- very stiff sand to clayey sand
- very stiff, fine grained

----> I_c/I_{SBT} plot s_u

—--< I_c/I_{SBT} plot D_r

EL : extremely low
VL : very low
L : low
M: medium
H : high

VL : very loose
L : loose
L : medium dense
D : dense
VD: very dense

Water Depth [m] : 30.0

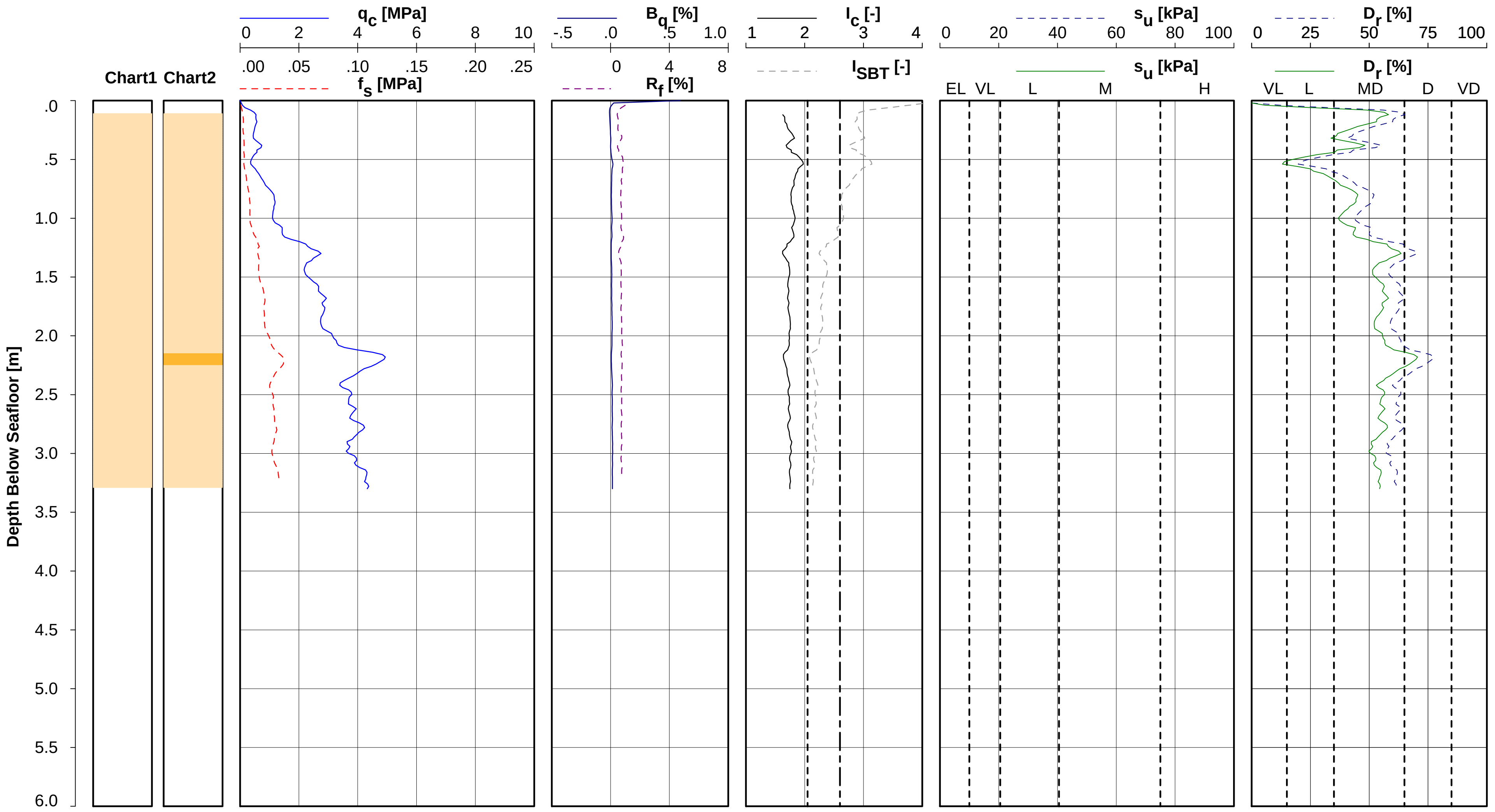
Coordinates [m] : E 490747 N 6025815

Notes:

- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT05



sensitive, fine grained

organic soils - PEATS

CLAYS - clay to silty clay

silt mixtures - clayey SILT to silty CLAY

SAND mixtures - silty sand to sandy silt

SANDS - clean sand to silty sand

gravelly sand to sand

very stiff sand to clayey sand

very stiff, fine grained

----> I_c/I_{SBT} plot s_u

—--< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 30.6

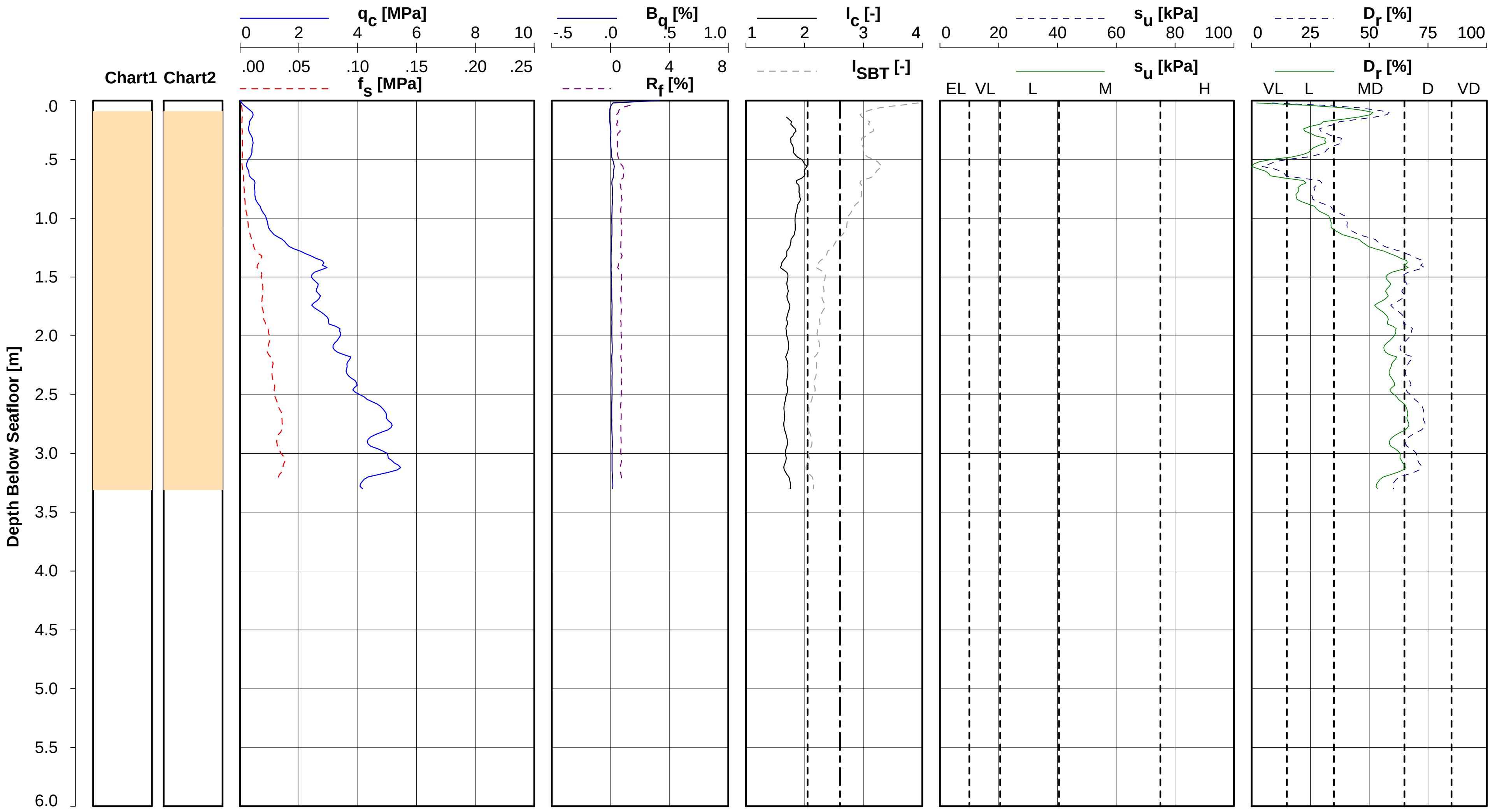
Coordinates [m] : E 491394 N 6025052

Notes:

- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT06



sensitive, fine grained

organic soils - PEATS

CLAYS - clay to silty clay

silt mixtures - clayey SILT to silty CLAY

SAND mixtures - silty sand to sandy silt

SANDS - clean sand to silty sand

gravelly sand to sand

very stiff sand to clayey sand

very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u

—— - - -< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

VL : very loose D : dense
L : loose VD: very dense
L : medium dense

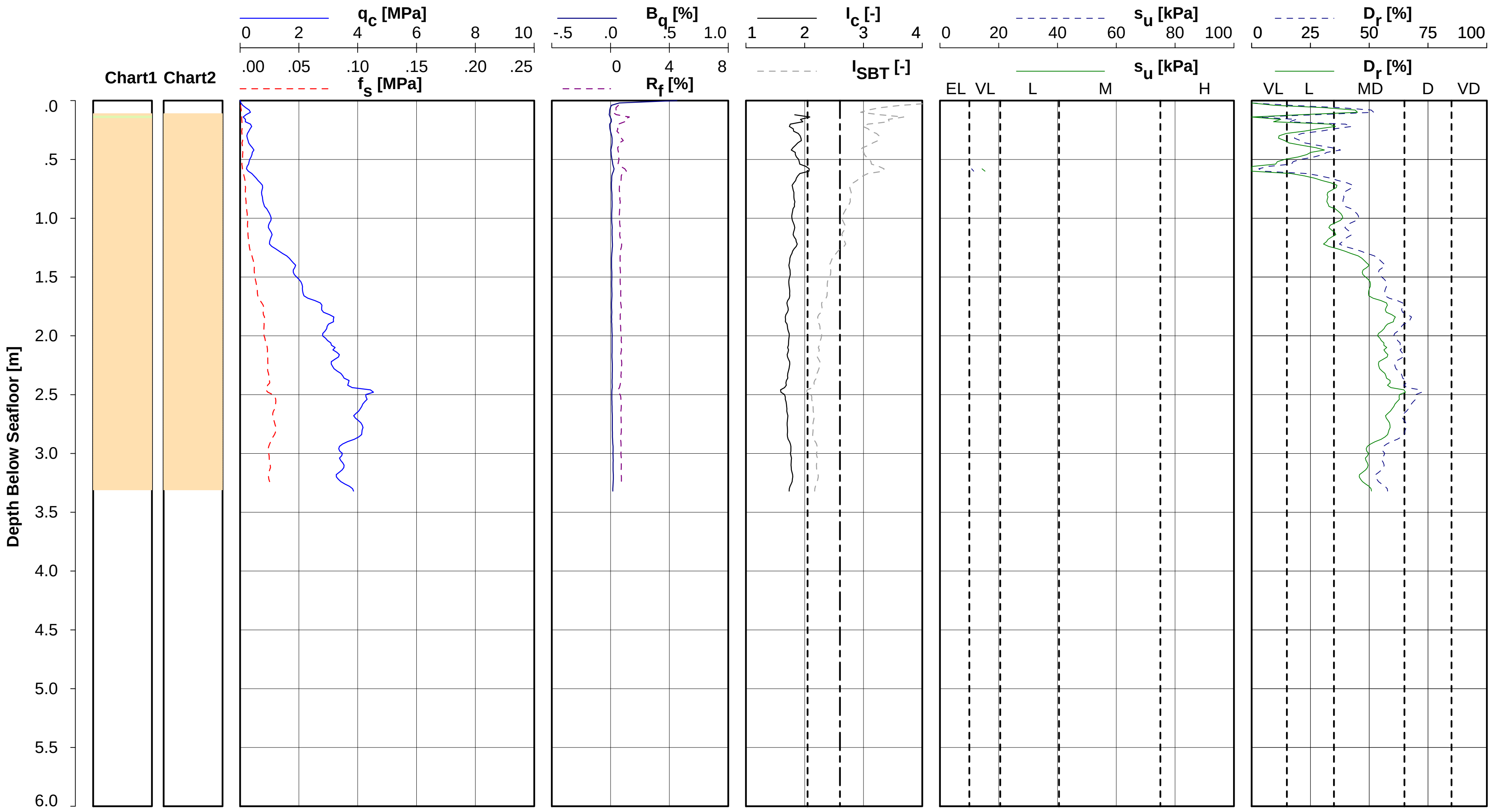
Water Depth [m] : 31.2 Coordinates [m] : E 492040 N 6024287

Notes:

- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT07



sensitive, fine grained

organic soils - PEATS

CLAYS - clay to silty clay

silt mixtures - clayey SILT to silty CLAY

SAND mixtures - silty sand to sandy silt

SANDS - clean sand to silty sand

gravelly sand to sand

very stiff sand to clayey sand

very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u

—— - - -< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

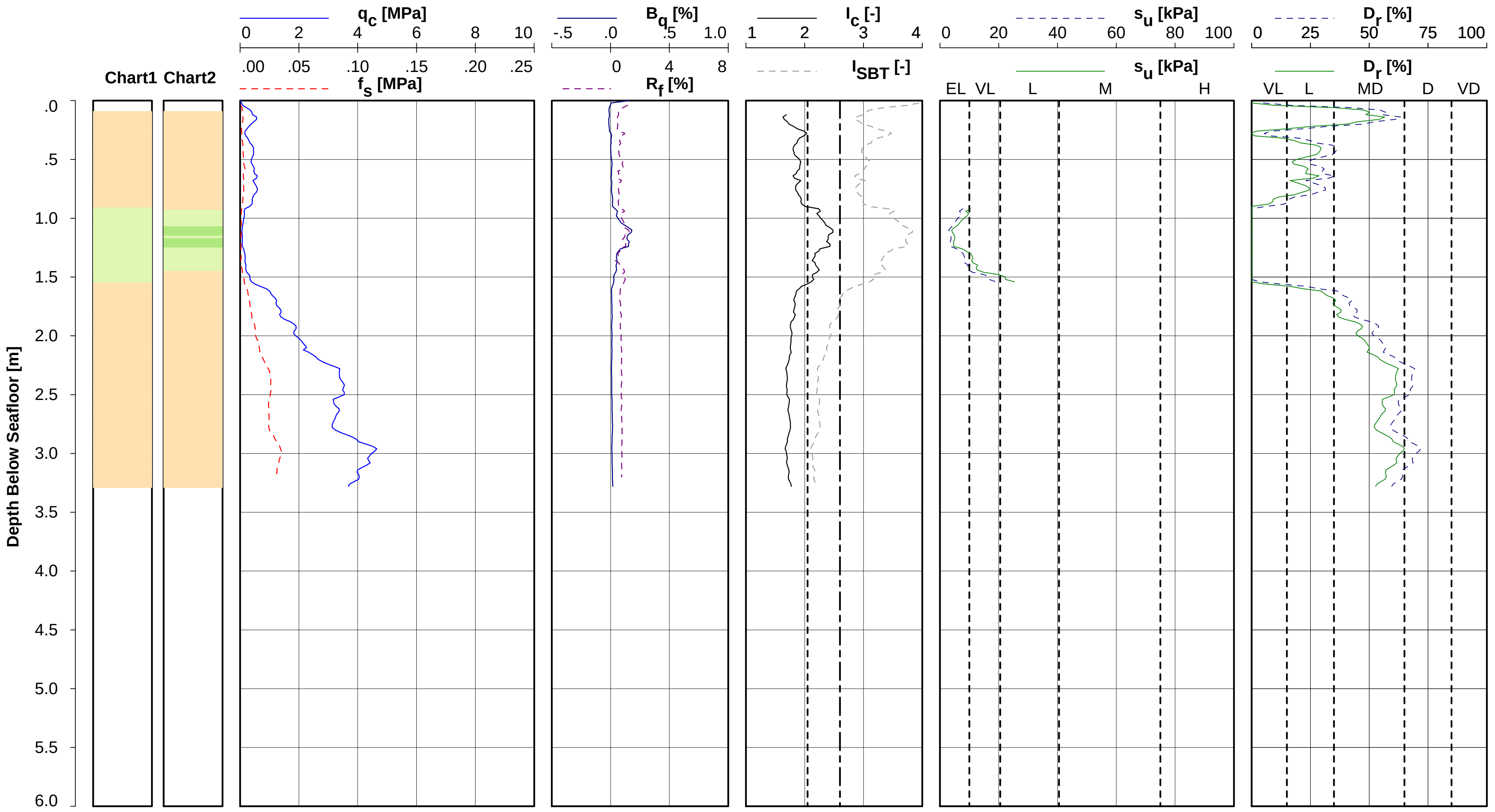
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 31.9 Coordinates [m] : E 492687 N 6023526

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT08



sensitive, fine grained

organic soils - PEATS

CLAYS - clay to silty clay

silt mixtures - clayey SILT to silty CLAY

SAND mixtures - silty sand to sandy silt

SANDS - clean sand to silty sand

gravelly sand to sand

very stiff sand to clayey sand

very stiff, fine grained

----> I_c/I_{SBT} plot s_u

—--< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

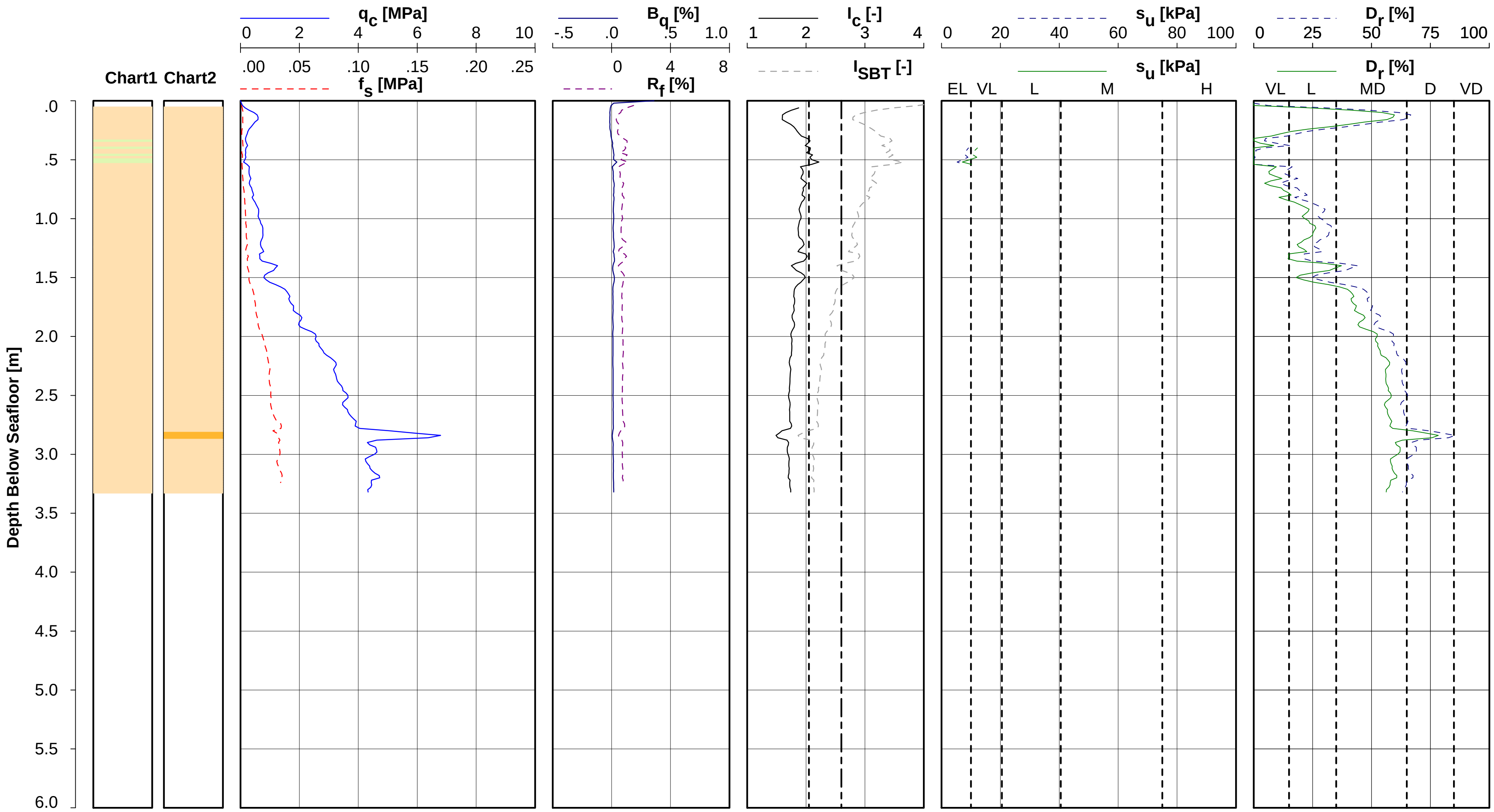
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 33.1 Coordinates [m] : E 493327 N 6022770

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT09



sensitive, fine grained
organic soils - PEATS
CLAYS - clay to silty clay
silt mixtures - clayey SILT to silty CLAY
SAND mixtures - silty sand to sandy silt
SANDS - clean sand to silty sand
gravelly sand to sand
very stiff sand to clayey sand
very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u
—— - - -< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

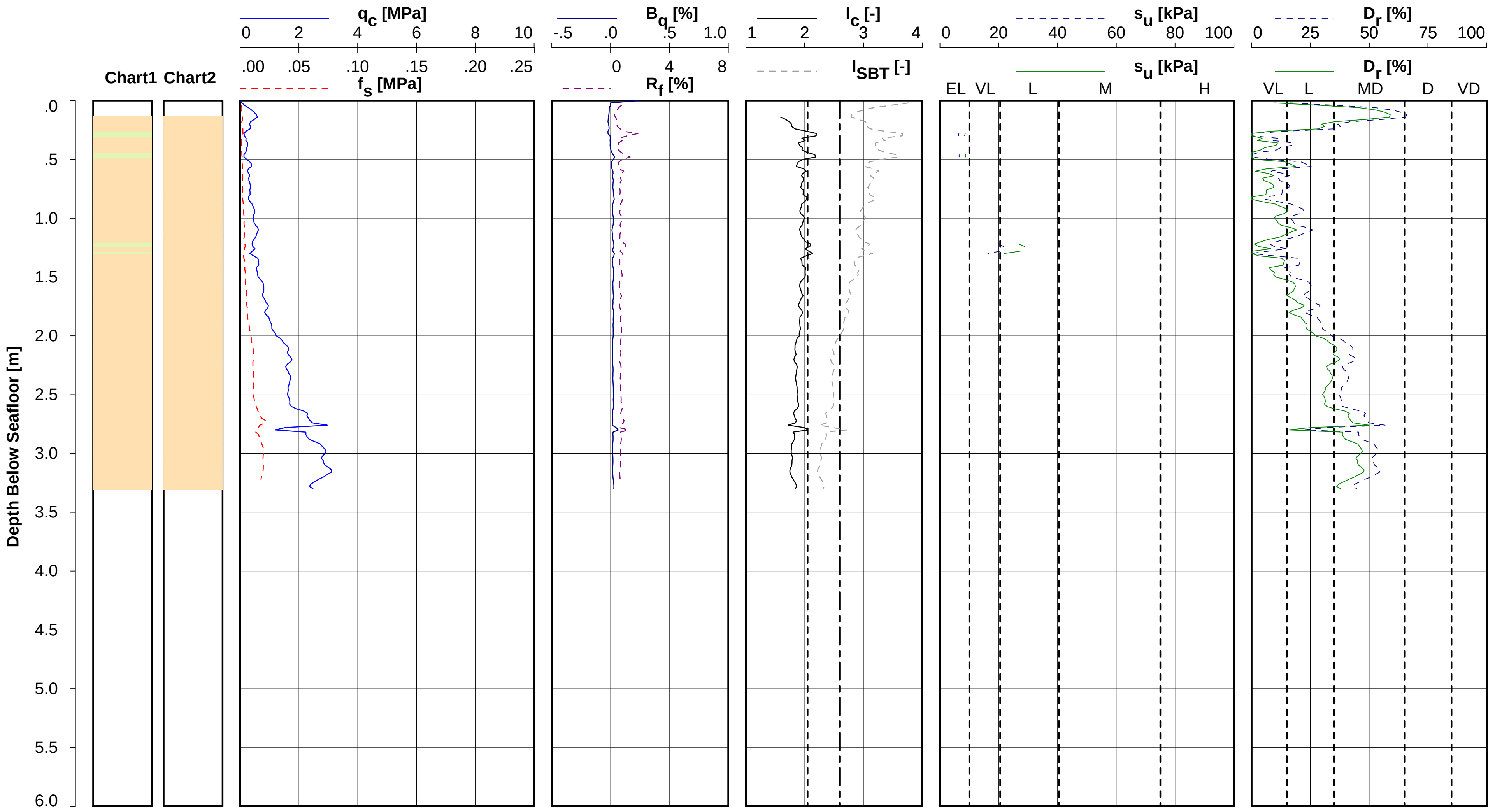
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 34.6 Coordinates [m] : E 493979 N 6021999

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT10



sensitive, fine grained
organic soils - PEATS
CLAYS - clay to silty clay
silt mixtures - clayey SILT to silty CLAY
SAND mixtures - silty sand to sandy silt
SANDS - clean sand to silty sand
gravelly sand to sand
very stiff sand to clayey sand
very stiff, fine grained

----> I_c/I_{SBT} plot s_u
—---< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

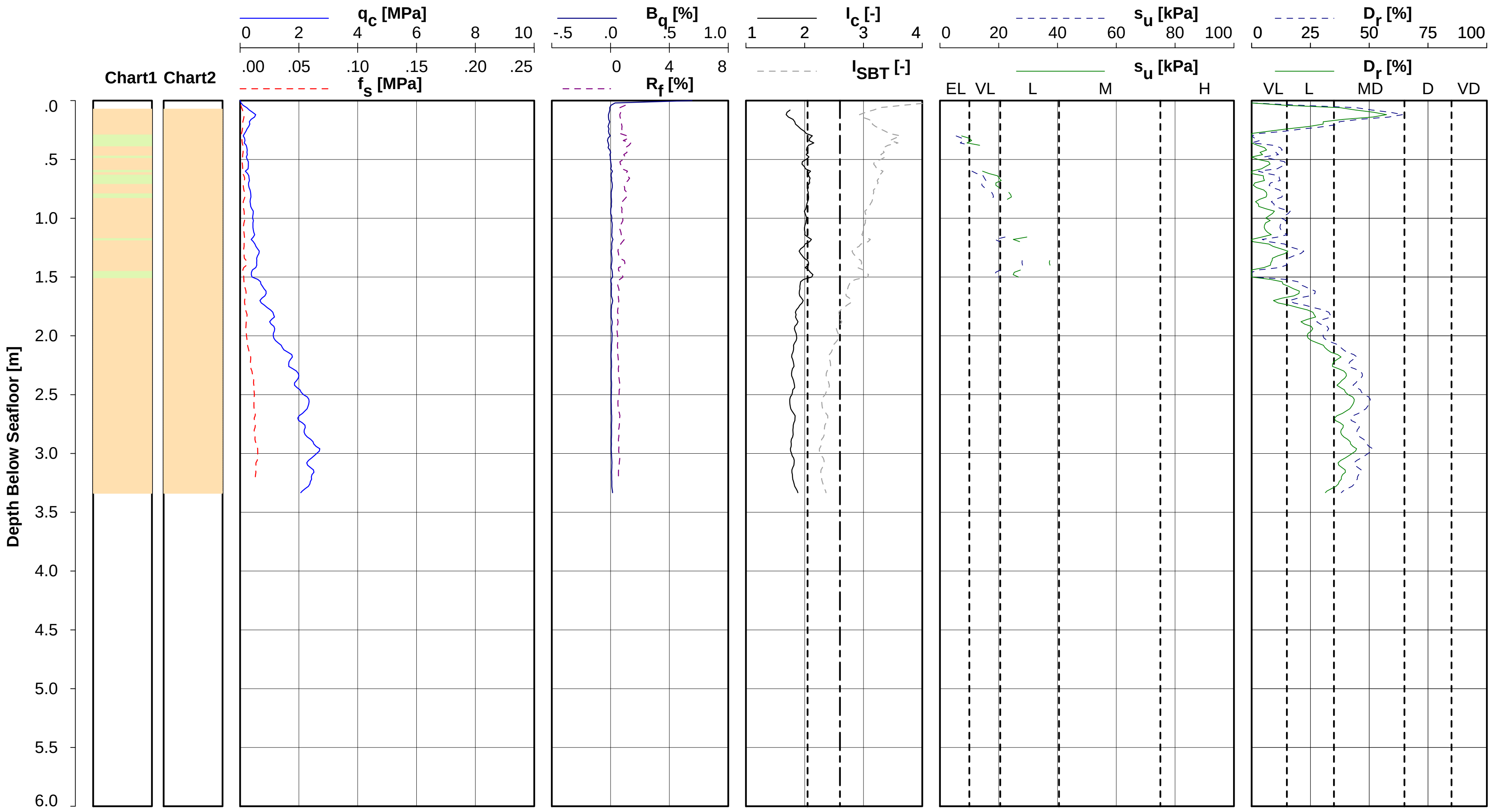
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 36.3 Coordinates [m] : E 494625 N 6021237

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT11



sensitive, fine grained

organic soils - PEATS

CLAYS - clay to silty clay

silt mixtures - clayey SILT to silty CLAY

SAND mixtures - silty sand to sandy silt

SANDS - clean sand to silty sand

gravelly sand to sand

very stiff sand to clayey sand

very stiff, fine grained

----> I_c/I_{SBT} plot s_u

—---< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

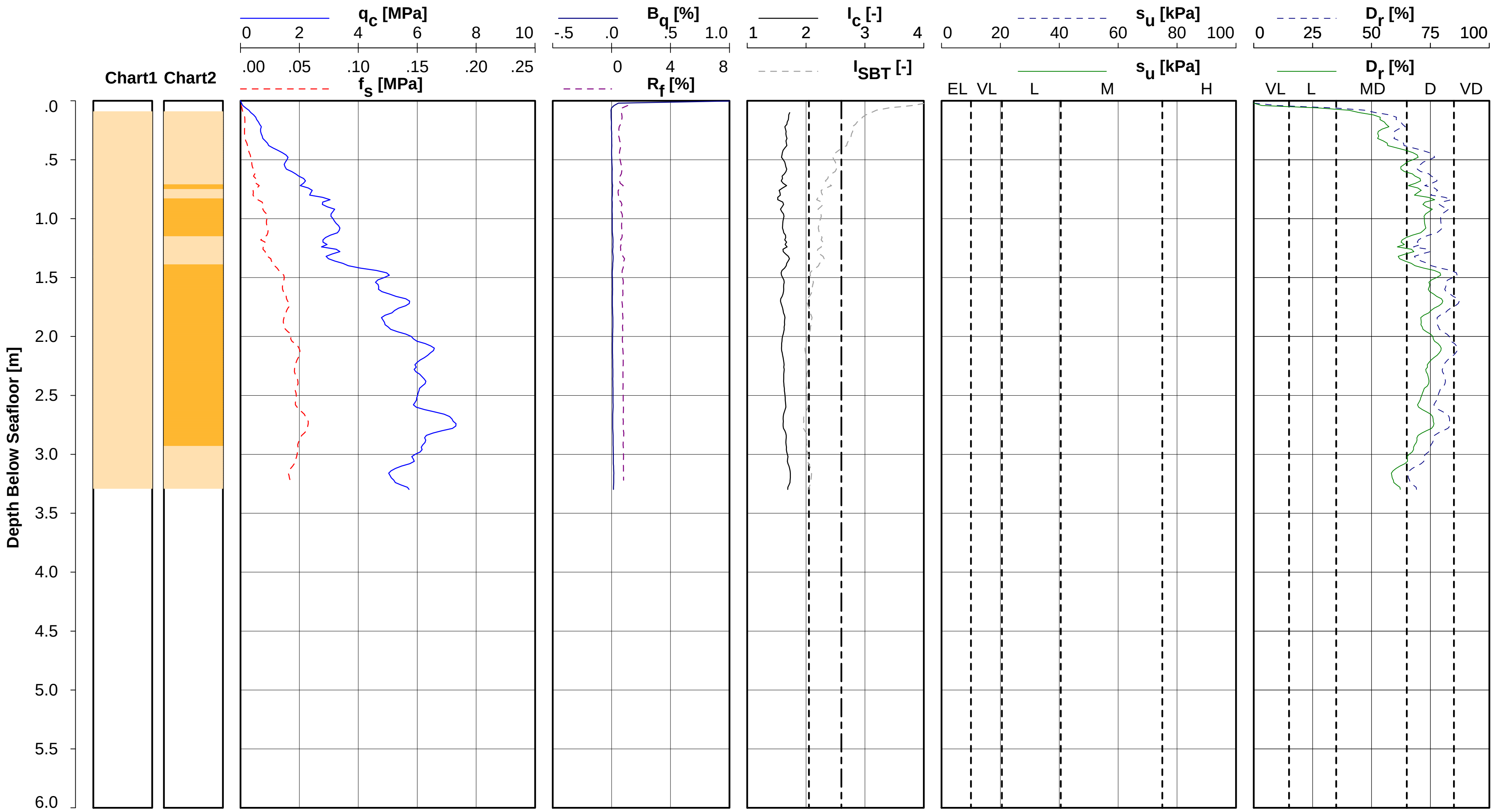
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 38.2 Coordinates [m] : E 495273 N 6020475

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT12



sensitive, fine grained

organic soils - PEATS

CLAYS - clay to silty clay

silt mixtures - clayey SILT to silty CLAY

SAND mixtures - silty sand to sandy silt

SANDS - clean sand to silty sand

gravelly sand to sand

very stiff sand to clayey sand

very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u
—— - - -< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

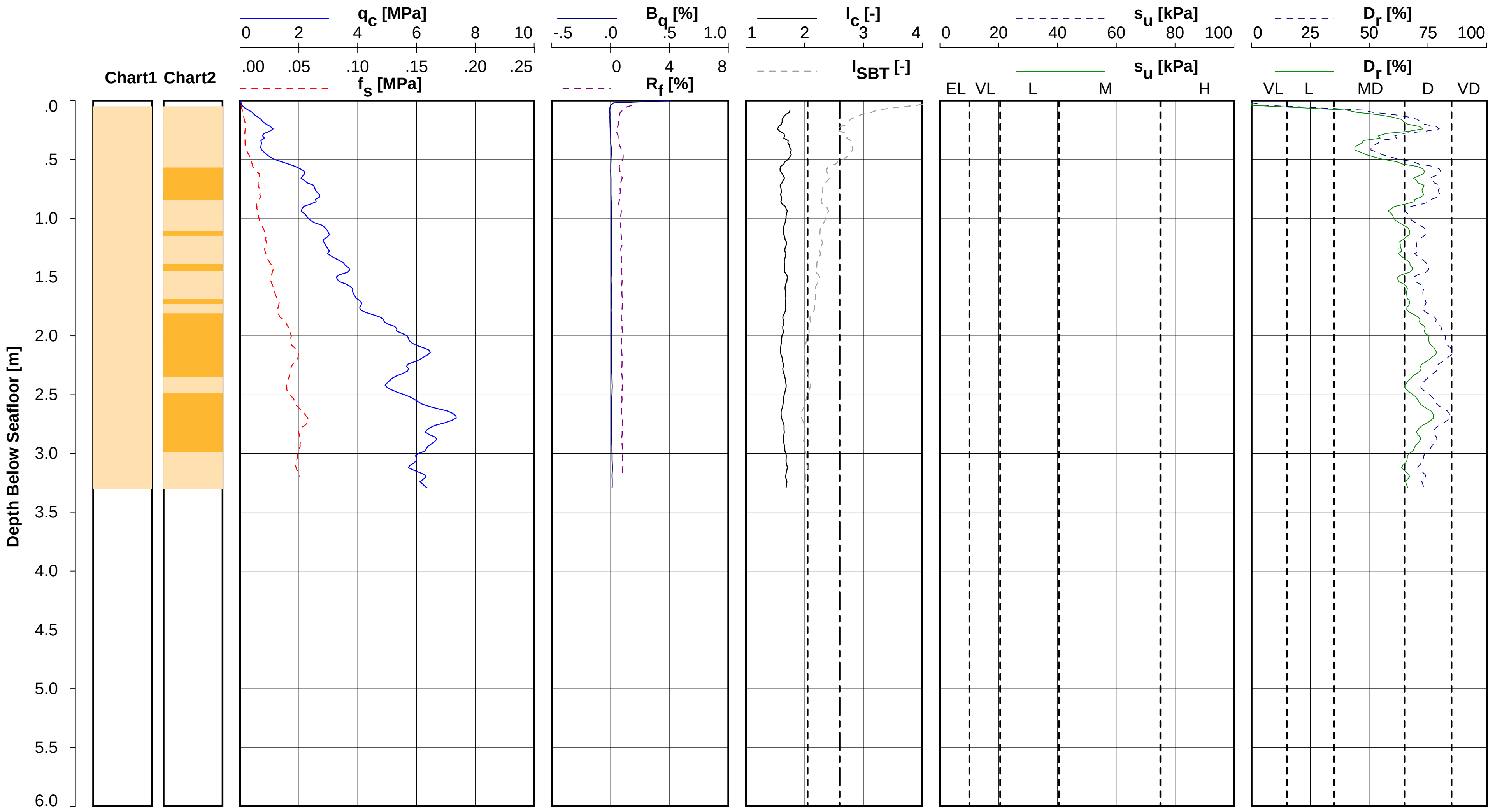
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 28.8 Coordinates [m] : E 488571 N 6027954

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT13



sensitive, fine grained

organic soils - PEATS

CLAYS - clay to silty clay

silt mixtures - clayey SILT to silty CLAY

SAND mixtures - silty sand to sandy silt

SANDS - clean sand to silty sand

gravelly sand to sand

very stiff sand to clayey sand

very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u
—— - - < I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

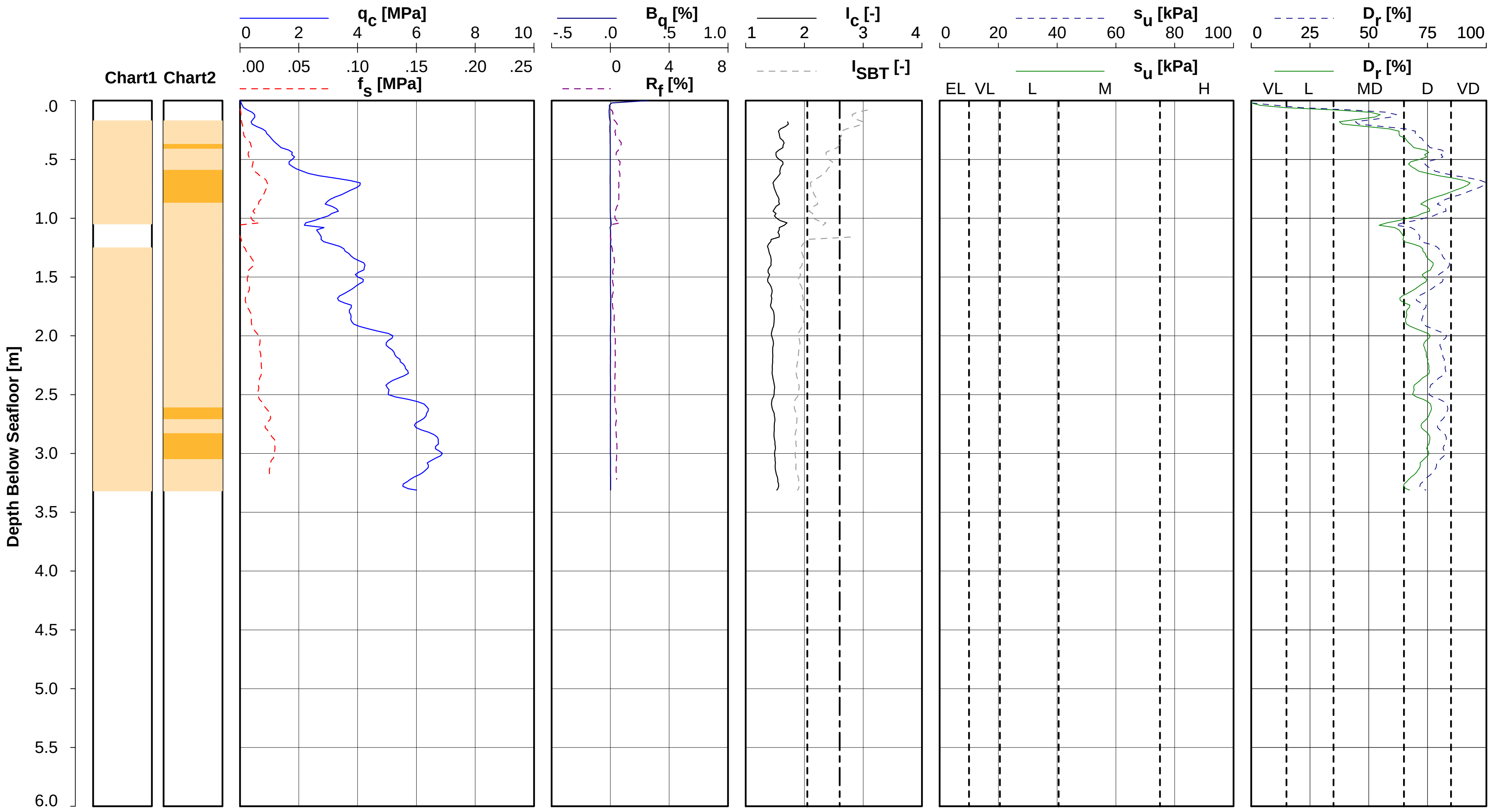
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 29.1 Coordinates [m] : E 488980 N 6027041

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT14



sensitive, fine grained
organic soils - PEATS
CLAYS - clay to silty clay
silt mixtures - clayey SILT to silty CLAY
SAND mixtures - silty sand to sandy silt
SANDS - clean sand to silty sand
gravelly sand to sand
very stiff sand to clayey sand
very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u
—- - -< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

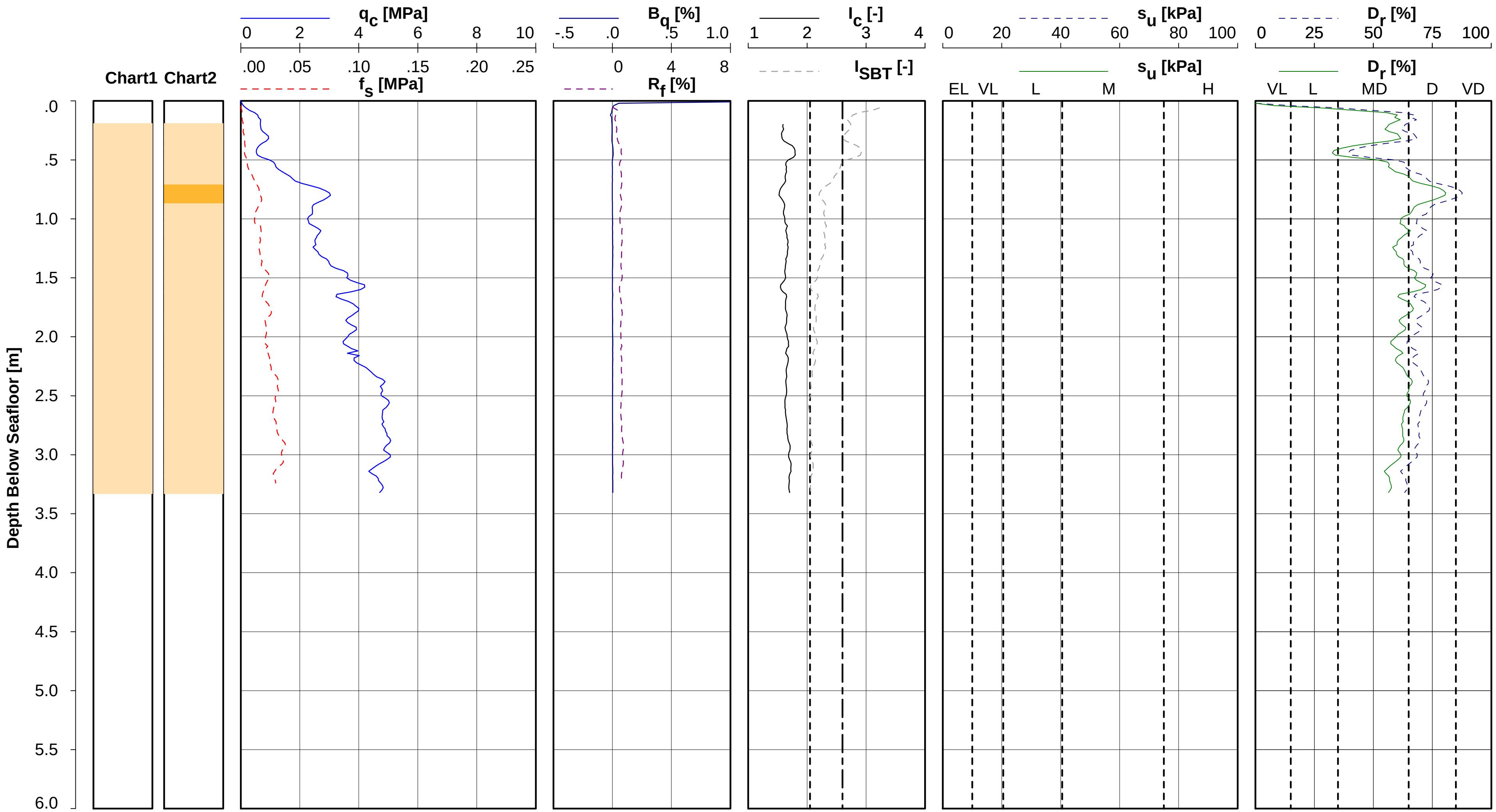
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 29.4 Coordinates [m] : E 489403 N 6026101

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT15



sensitive, fine grained
organic soils - PEATS
CLAYS - clay to silty clay
silt mixtures - clayey SILT to silty CLAY
SAND mixtures - silty sand to sandy silt
SANDS - clean sand to silty sand
gravelly sand to sand
very stiff sand to clayey sand
very stiff, fine grained

----> I_c/I_{SBT} plot s_u
—--< I_c/I_{SBT} plot D_r

EL : extremely low
VL : very low
L : low
M: medium
H : high

VL : very loose
L : loose
L : medium dense
D : dense
VD: very dense

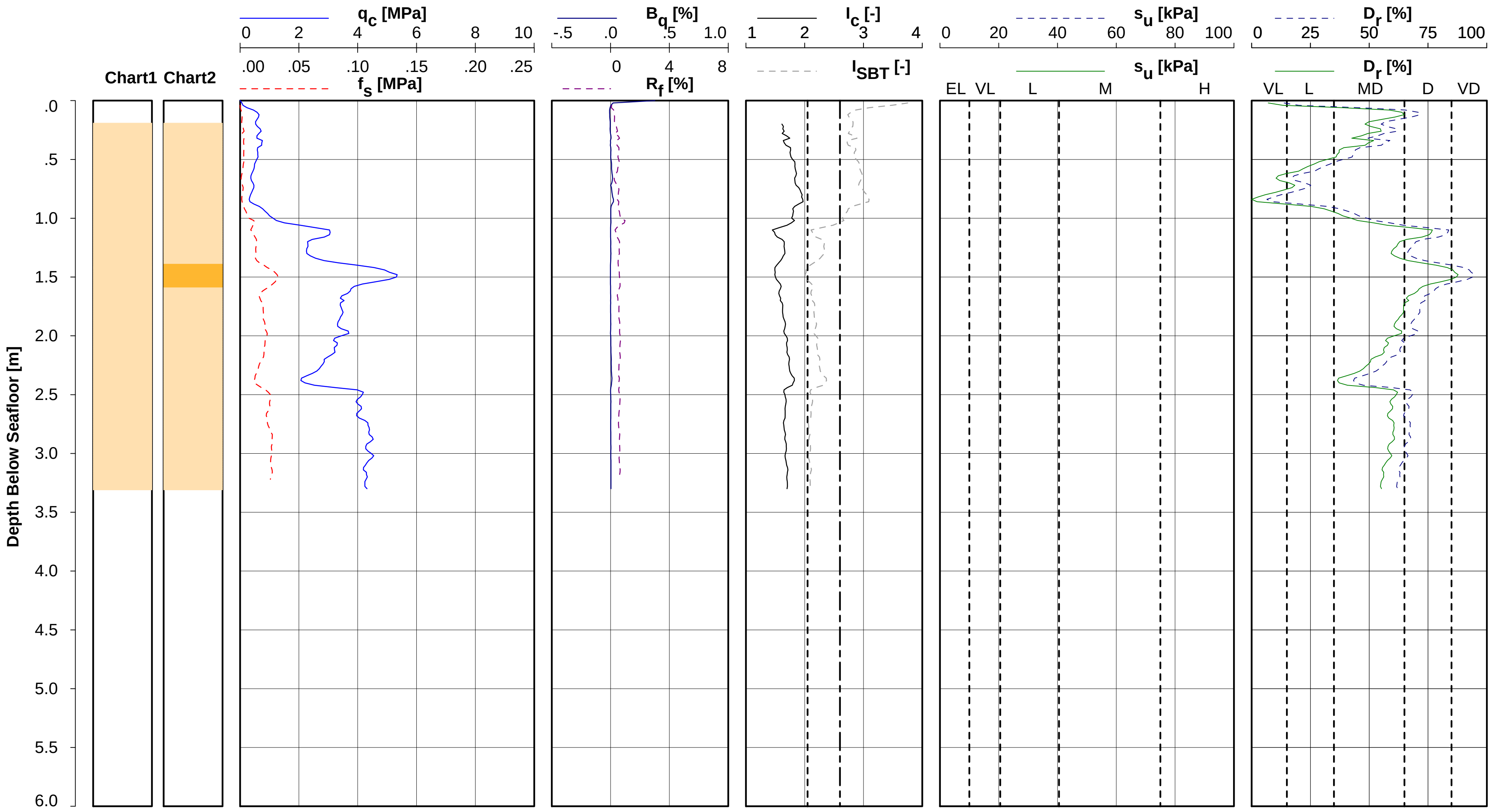
Water Depth [m] : 29.8

Coordinates [m] : E 489801 N 6025217

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT16



sensitive, fine grained
organic soils - PEATS
CLAYS - clay to silty clay
silt mixtures - clayey SILT to silty CLAY
SAND mixtures - silty sand to sandy silt
SANDS - clean sand to silty sand
gravelly sand to sand
very stiff sand to clayey sand
very stiff, fine grained

- - - -> I_c/I_{SBT} plot s_u
—— - - -< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

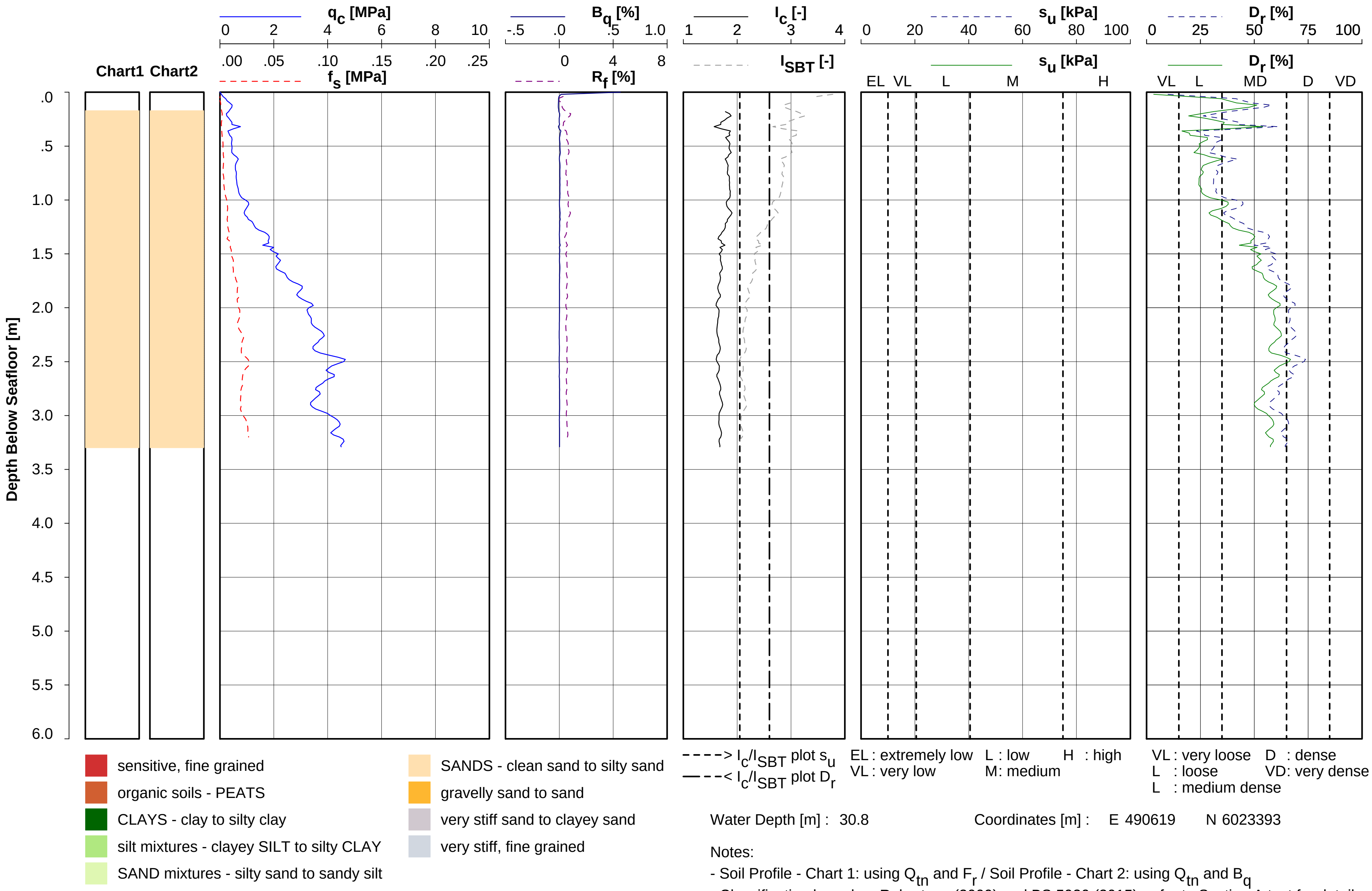
VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 30.3   Coordinates [m] : E 490211 N 6024306

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT17



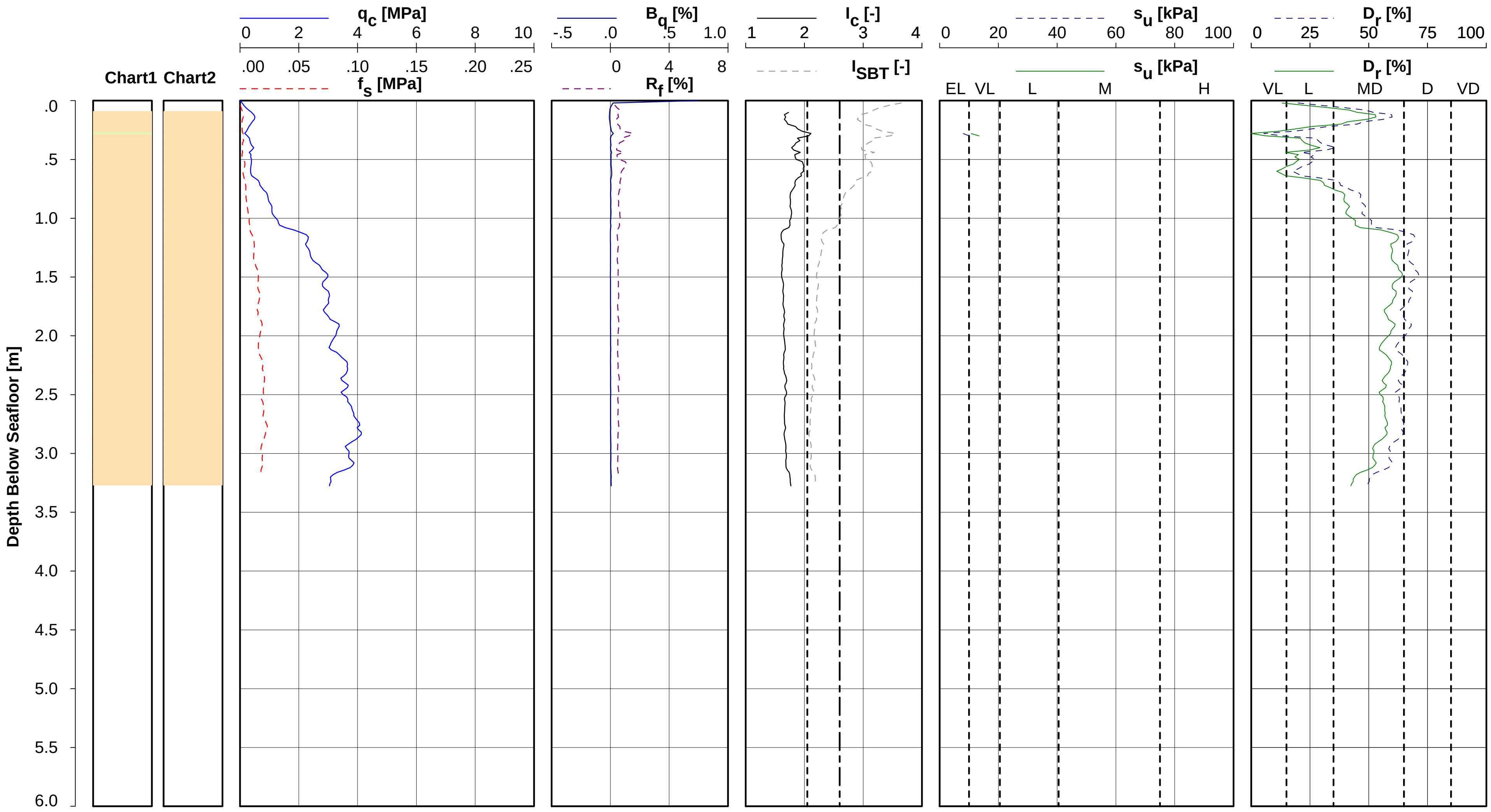
Water Depth [m] : 30.8 Coordinates [m] : E 490619 N 6023393

Notes:

- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT18



- sensitive, fine grained
- organic soils - PEATS
- CLAYS - clay to silty clay
- silt mixtures - clayey SILT to silty CLAY
- SAND mixtures - silty sand to sandy silt
- SANDS - clean sand to silty sand
- gravelly sand to sand
- very stiff sand to clayey sand
- very stiff, fine grained

----> I_c/I_{SBT} plot s_u
—--< I_c/I_{SBT} plot D_r

EL : extremely low L : low H : high
VL : very low M: medium

VL : very loose D : dense
L : loose VD: very dense
L : medium dense

Water Depth [m] : 31.4 Coordinates [m] : E 491029 N 6022481

Notes:
- Soil Profile - Chart 1: using Q_{tn} and F_r / Soil Profile - Chart 2: using Q_{tn} and B_q
- Classification based on Robertson (2009) and BS 5930 (2015), refer to Section A text for details
- Soil Behaviour Type Index (I_c and I_{SBT}) based on Robertson (1990) and Robertson (2010)

CONE PENETRATION TEST INTERPRETATION (ROBERTSON)

CPT19

SECTION B: IN SITU TEST RESULTS

TEXT – SECTION B: Page

LIST OF PLATES IN SECTION B: Plate

B. IN SITU TEST RESULTS

B.1 PRACTICE FOR CONE PENETRATION TEST

Test Overview

General Procedure:	– According to ISO (2014) for test practice – According to ISO (2003) for metrological confirmation – Refer to document titled “Cone Penetration Test” presented in Appendix 1
Target Application Class:	Class 3 of ISO (2014), refer to document titled “Cone Penetration Test” presented in Appendix 1
Set-up Stage:	Location as directed by Client
Additional Measurements:	Not applicable
Test Stage:	Not applicable
Test Termination:	Refer to document titled “Cone Penetration Test” presented in Appendix 1
Drill-Out:	Not applicable
Test Site Restoration:	– No backfill of test hole – Local seabed disturbance – Possibility of local seafloor depression(s)

CPT Apparatus

Thrust Machine:	SEASCOUT® hydraulic jacking unit of nominal 10 kN thrust capacity and 5 m stroke
Reaction Equipment:	SEASCOUT 10 seabed frame
Push Rods:	25.3 mm push rod outer diameter (CP5)
Push Rod Casing:	Not applicable
Penetrometer Type:	Type CP5-CF35 Friction-cone penetrometer, 35 kN load sensors, 25 MPa pressure sensor, HDPE filter in cylindrical extension above base of cone, with non-directional inclinometer, 500 mm^2 cone base area, 7,500 mm^2 sleeve area

Test Results

Data Processing and Management:	– Refer to document titled “Cone Penetration Test” presented in Appendix 1 – UNIPLOT software – Graphical scales selected to suit general presentation of data and requirements of standards, where practicable – No display of data outside of chart limits, i.e. some values may not be shown
Data Format(s):	– PDF for viewing and printing (this primary document)
Water Depth Reference:	As obtained from multi beam echo sounder readings at prior to start of testing
Depth Reference Level:	Seafloor, particularly: – No depth reference correction applied for depth below seafloor – No evidence for extremely soft ground at seafloor but no specific measurements performed – Base of seabed frame assumed level with seafloor at start of testing – Depth accuracy assessment of “Seabed – Favourable”; refer to document titled Positioning Survey and Depth Measurement presented in Appendix 1

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Depth Correction for Penetrometer Inclination:	Applicable
Parameter Values for Data Processing:	Refer to plate(s) titled "Cone Penetration Test Results" showing soil behaviour type index, soil unit weight and supplementary normalised parameter values

B.2 COMMENTS ON RESULTS

A value of 10 kN/m^3 was used as an average density of seawater and drill fluid to calculate the theoretical hydrostatic pressure at the seabed level. The hydrostatic pressure is used for comparison with the cone measurements prior to testing and when calculating total cone resistance.

A unit weight of 19.5 kN/m^3 was considered for the different soil units encountered. Selected value was based on available geotechnical investigation data applicable to nearby sites.

Deck measurements, referred to as "zero reading", were taken before the start and after the end of each test to check the cone performance during testing and verify equipment conditions during test deployment. Plates titled "Cone Penetration Test: Zero Load Drift" include the results of this comparison. Zero load verification showed indicates satisfactory cone performance during testing.

At location CPT15, it appears that the cone pushes against a cobble/boulder or penetrates through a gravelly zone around ca. 1.1 m BSF. A shift (drift) in friction and possibly cone measurements is apparent. This is not uncommon for testing such formations. The drift in values is permanent, i.e. values after testing on deck (zero loads) show a shift. Selected offset of friction considered the measurements before the observed drift in friction. Friction and cone tip values below a depth of 1.1m BSF appear valid and in line with expectation for these soils. However, sleeve friction and cone tip measurements should be used with caution considering the uncertainty in test reference level following the measurement shift.

Target penetration depth was 3 m BSF. This was achieved at all locations apart for CPT01. At location CPT01 1.75 m penetration was achieved. The test was stopped early to prevent damage to the test equipment following an observed rapid increase of cone inclination.

B.3 REFERENCES

- ASTM International, 2012. *ASTM D5778-12 Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils.* West Conshohocken: ASTM International.
- Computer Program UNIPLOT, *Processing of CPT data*
- International Organization for Standardization, 2003. *ISO 10012:2003 Measurement Management Systems - Requirements for Measurement Processes and Measuring Equipment*. Geneva: ISO.
- International Organization for Standardization, 2012. *ISO 22476-1:2012 Geotechnical Investigation and Testing – Field Testing – Part 1: Electrical Cone and Piezocone Penetration Tests.* Geneva: ISO. (With Technical Corrigendum 1, January 2013).
- International Organization for Standardization, 2014. *ISO 19901-8:2014 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures – Part 8: Marine Soil Investigations.* Geneva: ISO.

- Mayne, P.W., Peuchen, J. and Bouwmeester, D. 2010. Soil Unit Weight Estimated from CPTu in Offshore Soils. In Gourvenec, S. and White, D. Eds., *Frontiers in Offshore Geotechnics II: Proceedings of the 2nd International Symposium on Frontiers in Offshore Geotechnics, Perth, Australia, 8-10 November 2010,* Boca Raton: CRC Press, pp. 371-376.
- Peuchen, J., Vanden Berghe, J.F. and Coulais, C. 2010. Estimation of u_1/u_2 Conversion Factor for Piezocone. In *CPT'10: 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA: Conference Proceedings.*

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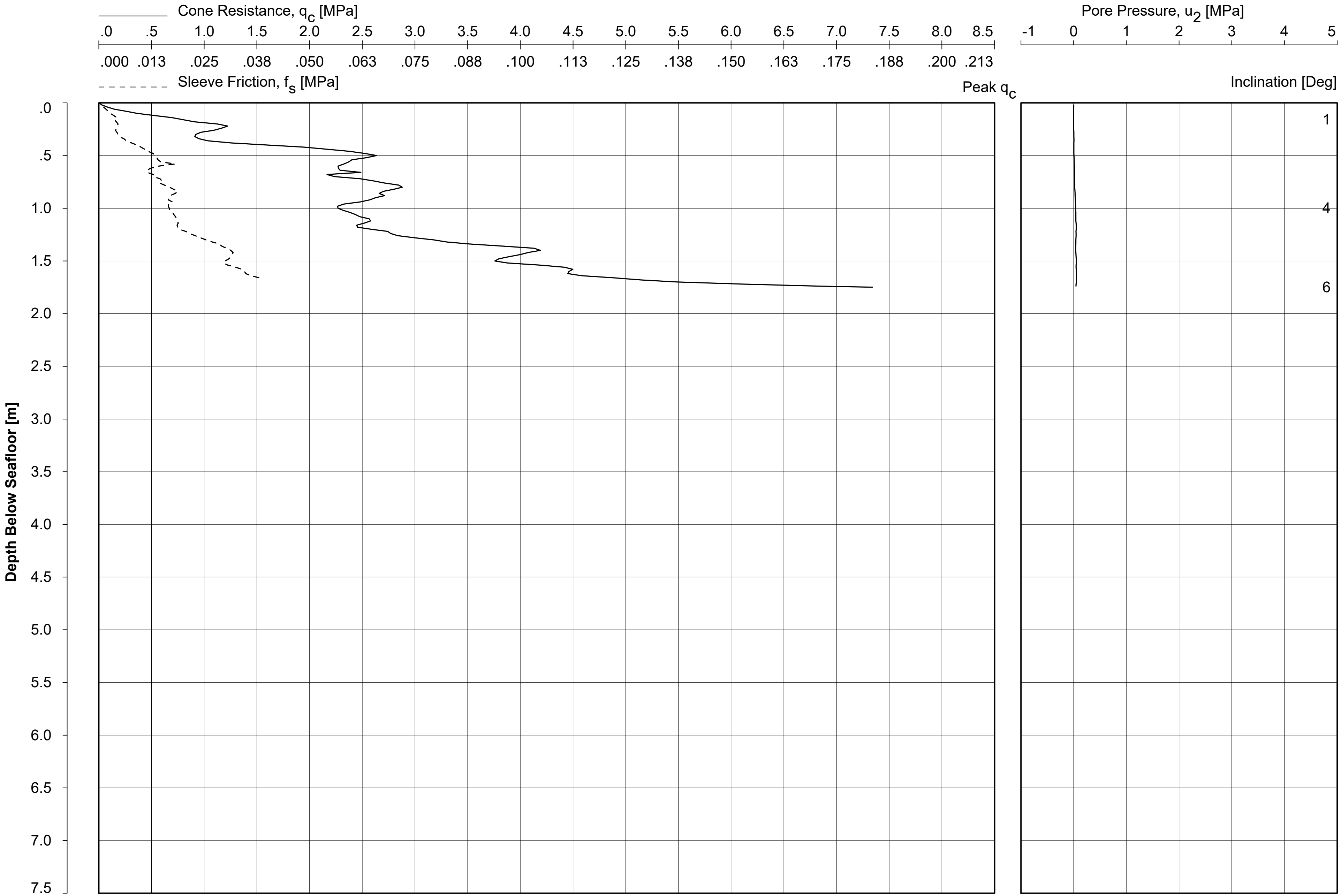

Date Of Testing : 19-Apr-2017
Water Depth [m] : 28.6
Coordinates [m] : E488162 N6028866

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT01

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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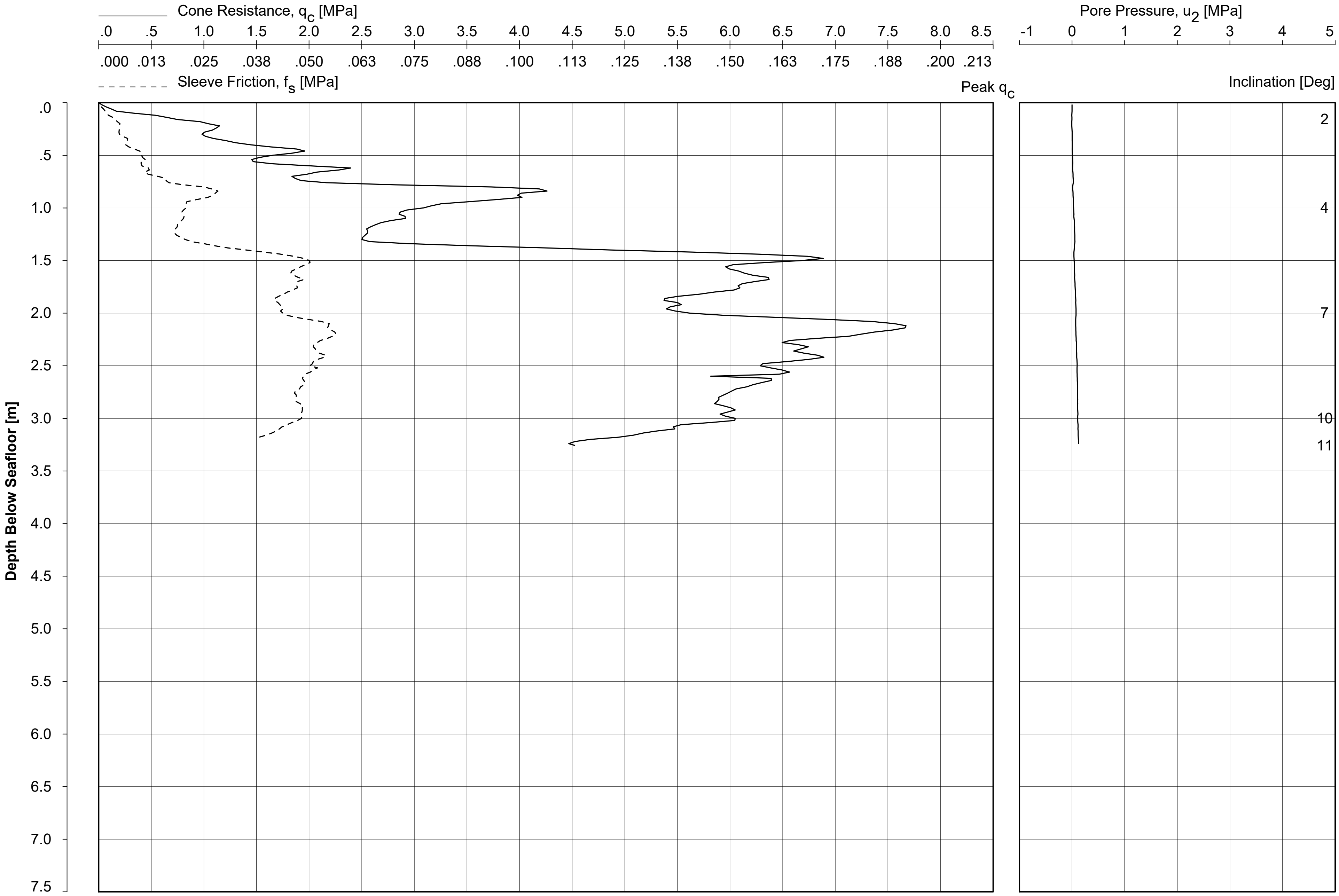


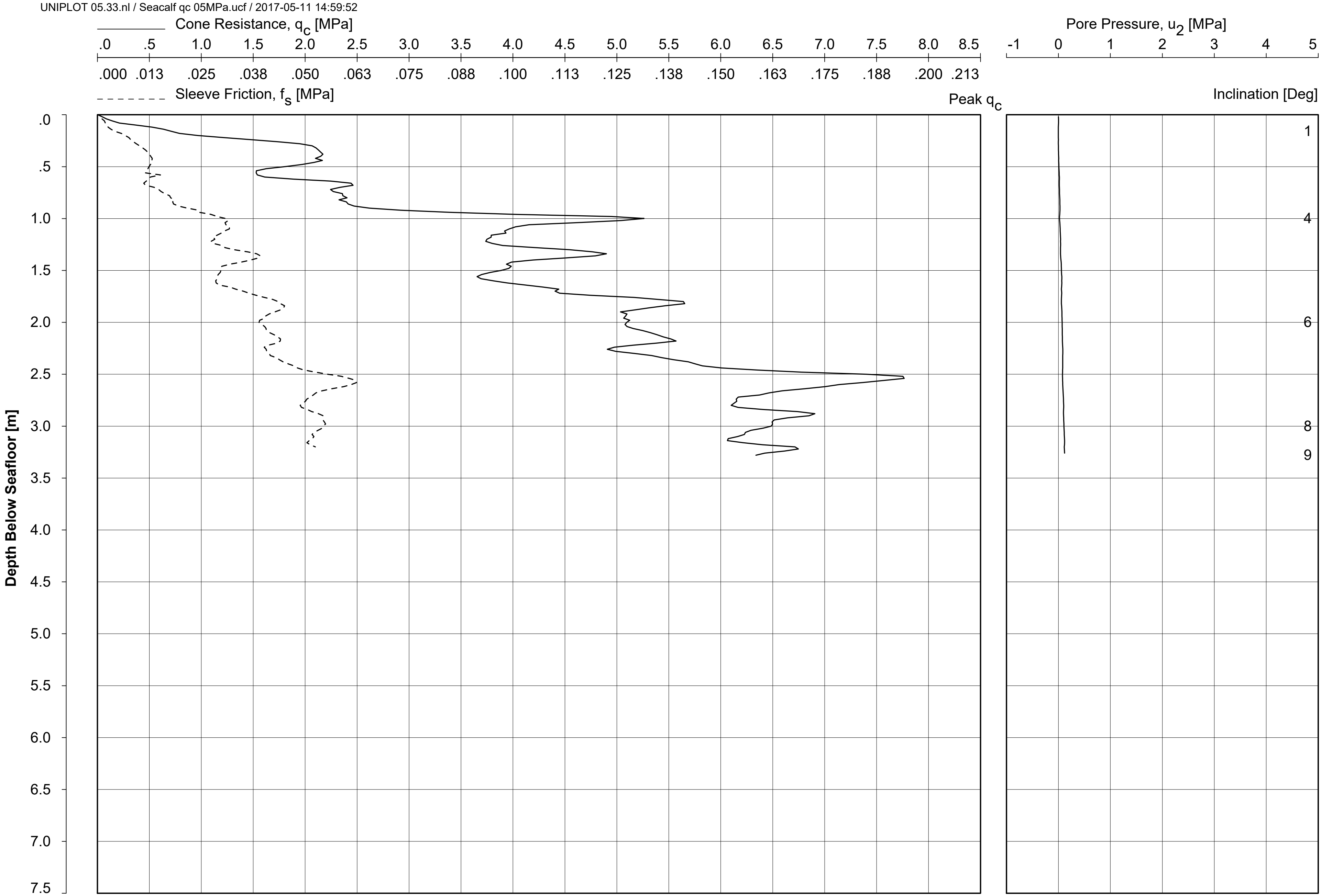
Date Of Testing : 19-Apr-2017
Water Depth [m] : 28.9
Coordinates [m] : E488807 N6028105

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT02

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA



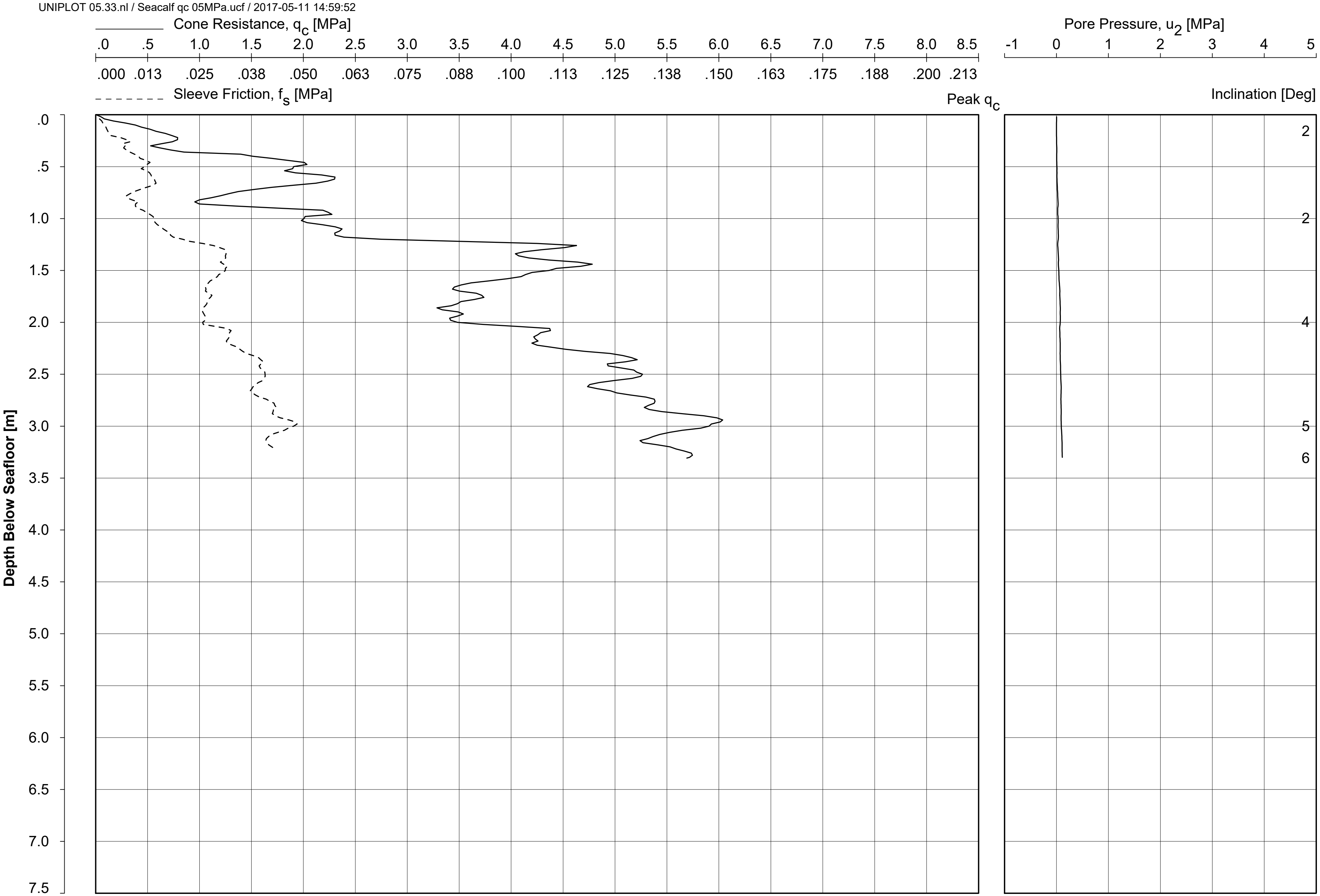
Date Of Testing : 19-Apr-2017
Water Depth [m] : 29.3
Coordinates [m] : E489454 N6027342

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT03

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA



Date Of Testing : 19-Apr-2017
Water Depth [m] : 29.6
Coordinates [m] : E490101 N6026576

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT04

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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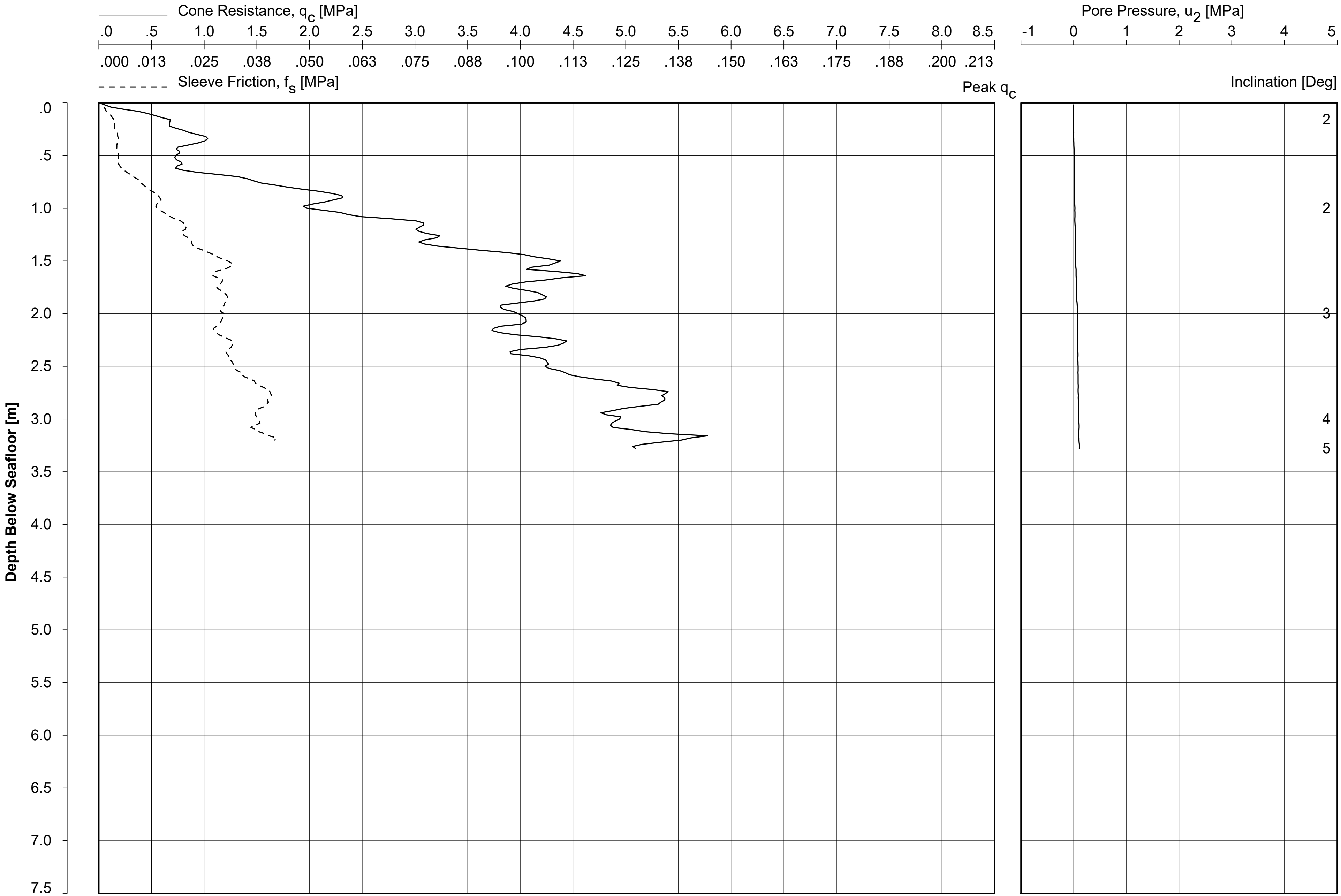

Date Of Testing : 19-Apr-2017
Water Depth [m] : 30.0
Coordinates [m] : E490747 N6025815

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT05

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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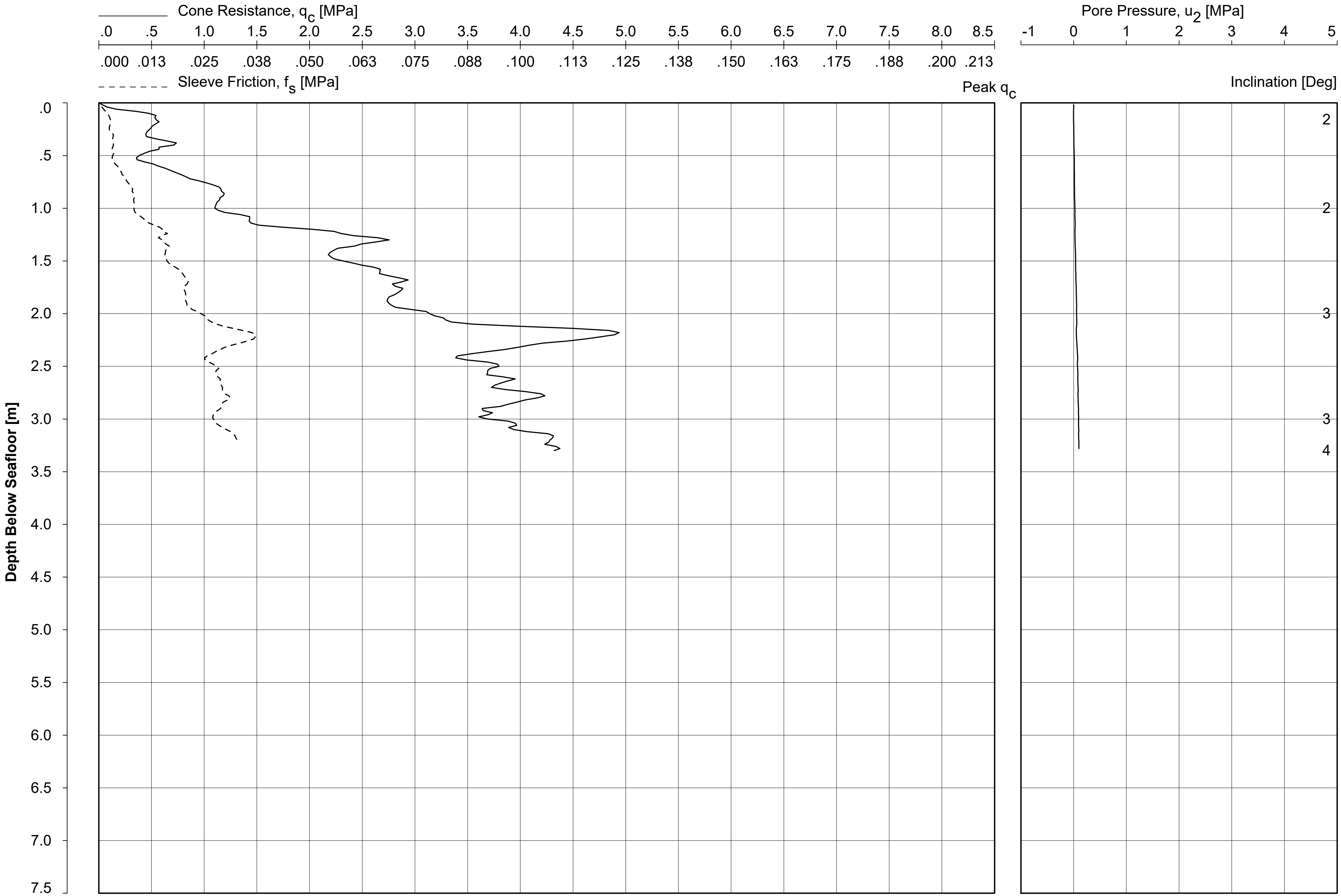

Date Of Testing : 19-Apr-2017
Water Depth [m] : 30.6
Coordinates [m] : E491394 N6025052

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT06

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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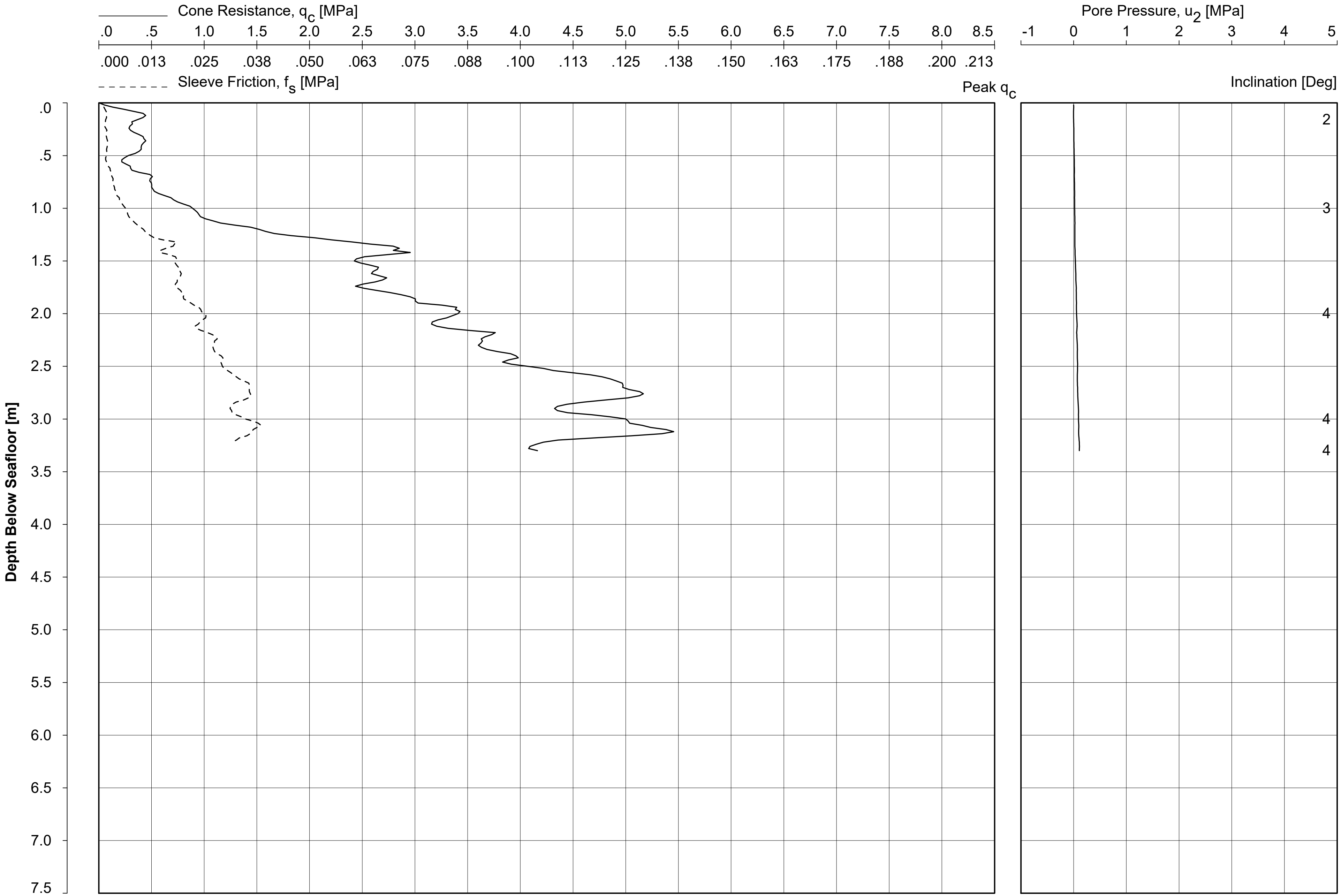

Date Of Testing : 19-Apr-2017
Water Depth [m] : 31.2
Coordinates [m] : E492040 N6024287

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT07

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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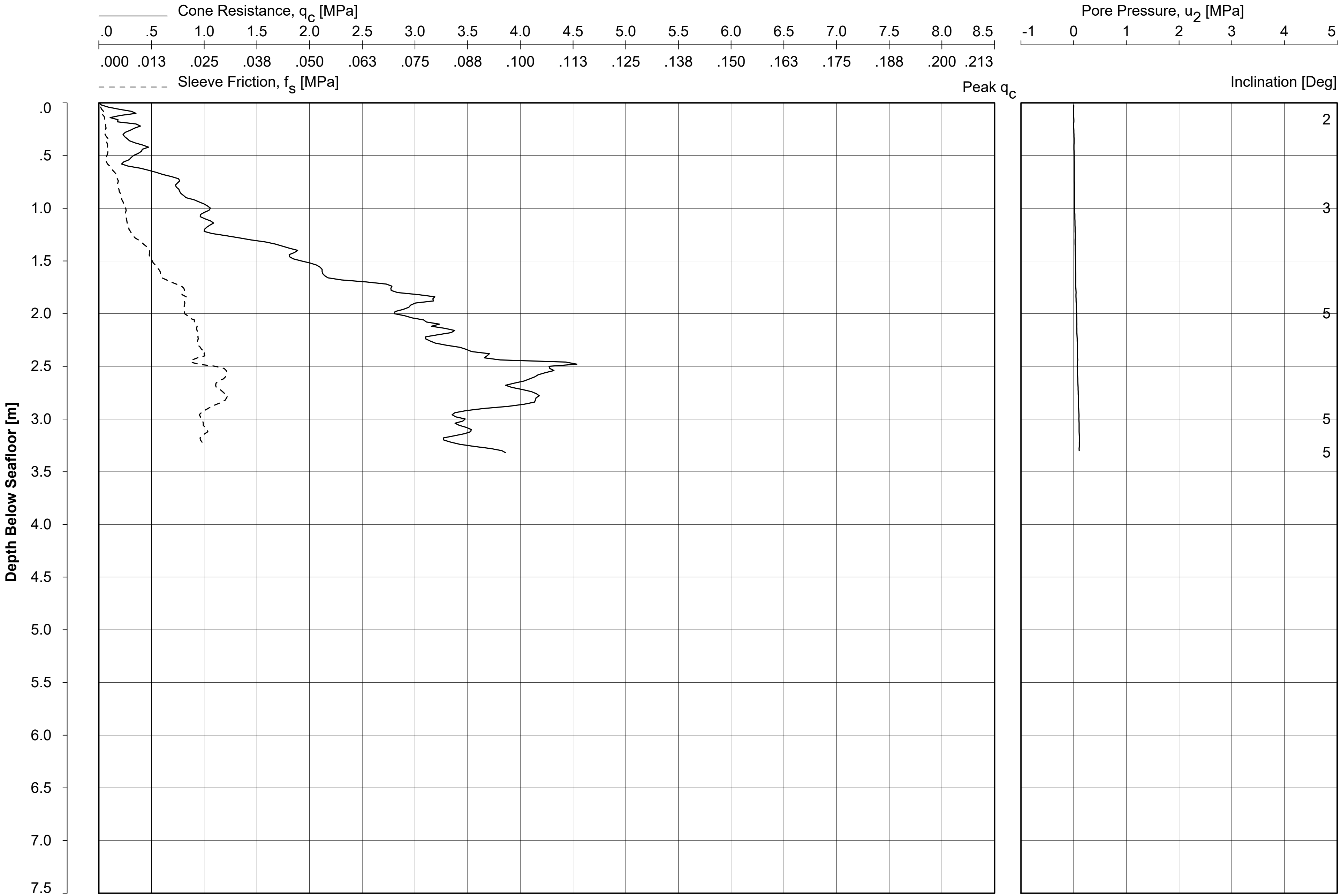

Date Of Testing : 19-Apr-2017
Water Depth [m] : 31.9
Coordinates [m] : E492687 N6023526

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT08

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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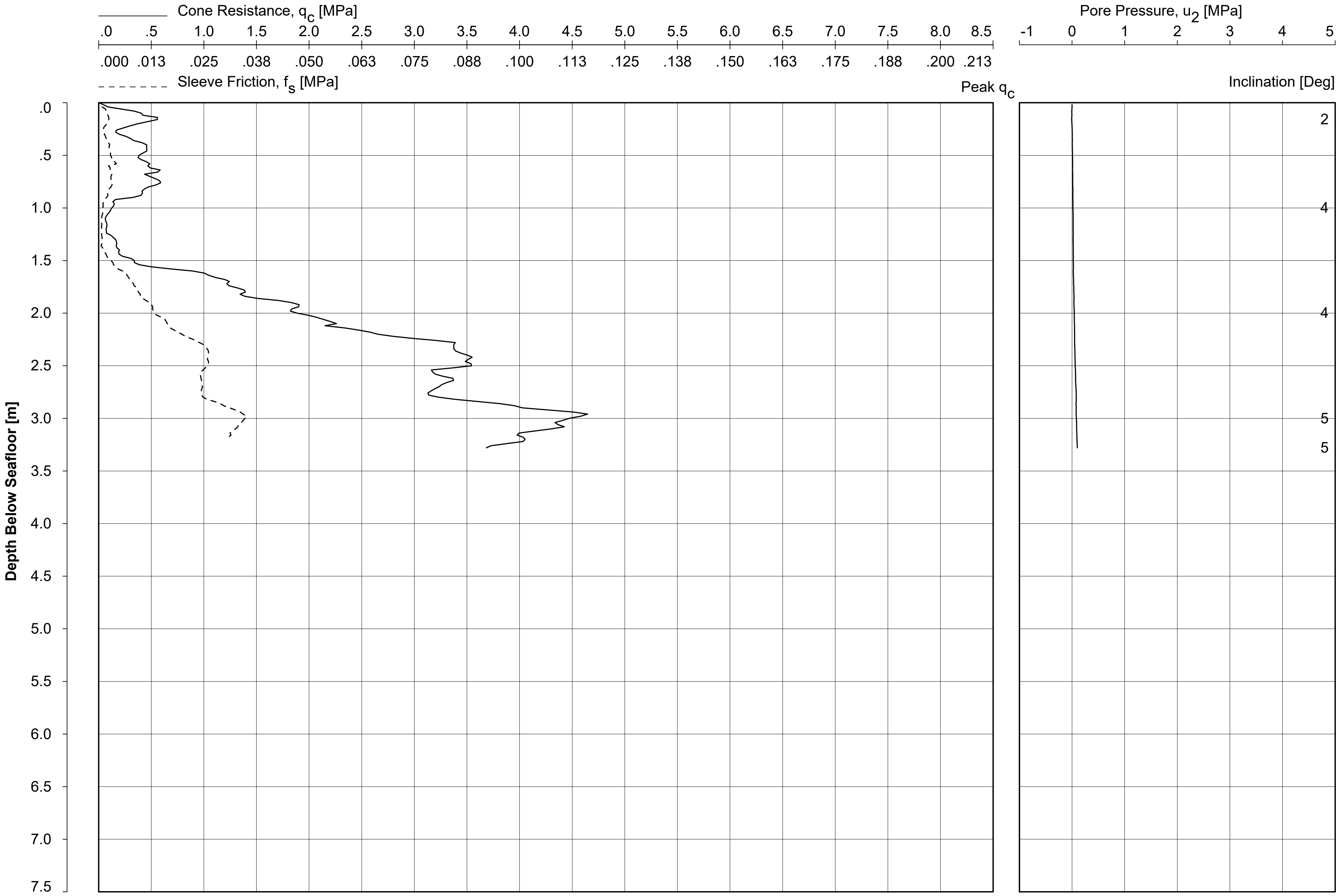


Date Of Testing : 19-Apr-2017
Water Depth [m] : 33.1
Coordinates [m] : E493327 N6022770

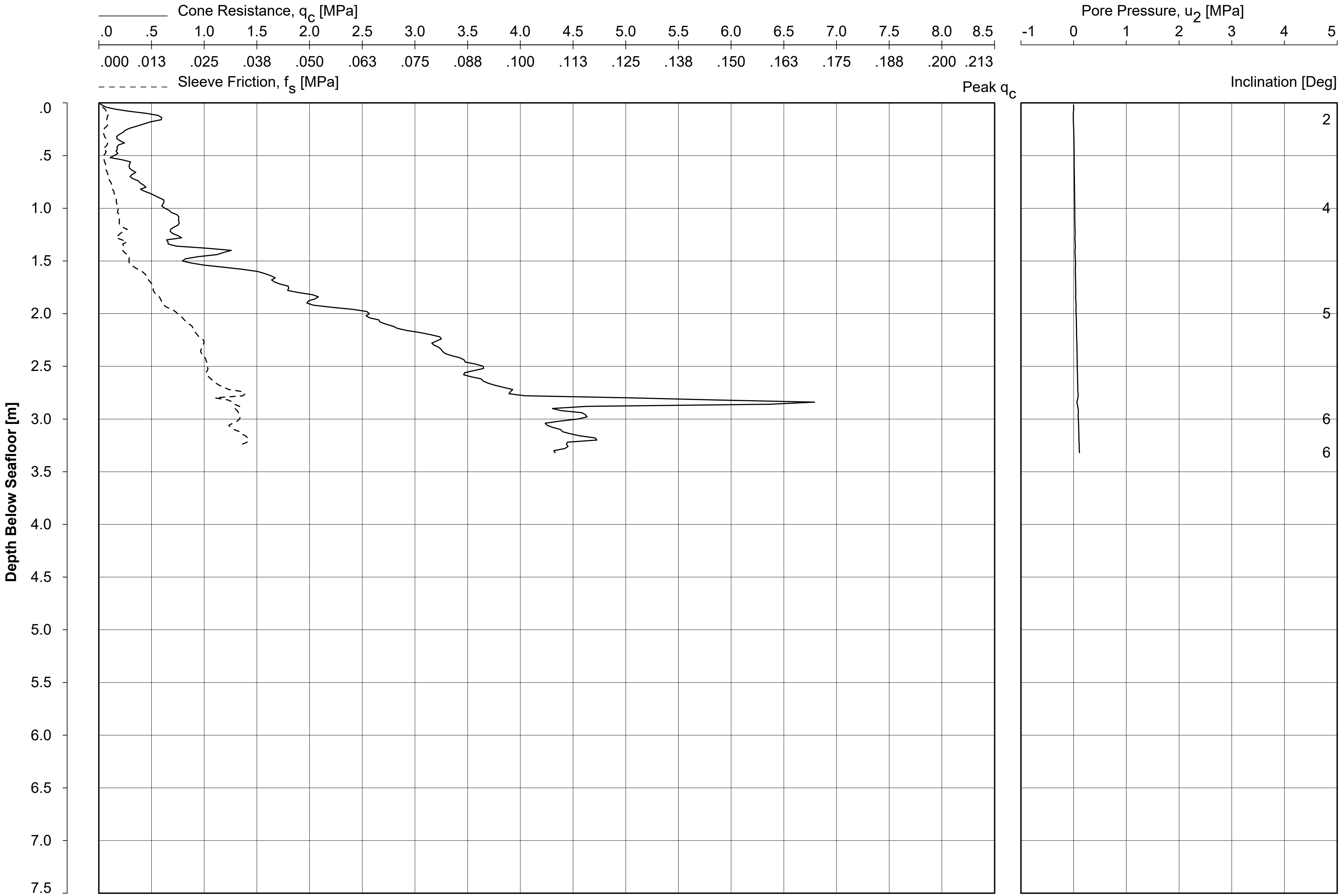
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT09

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:52



Date Of Testing : 19-Apr-2017
Water Depth [m] : 34.6
Coordinates [m] : E493979 N6021999

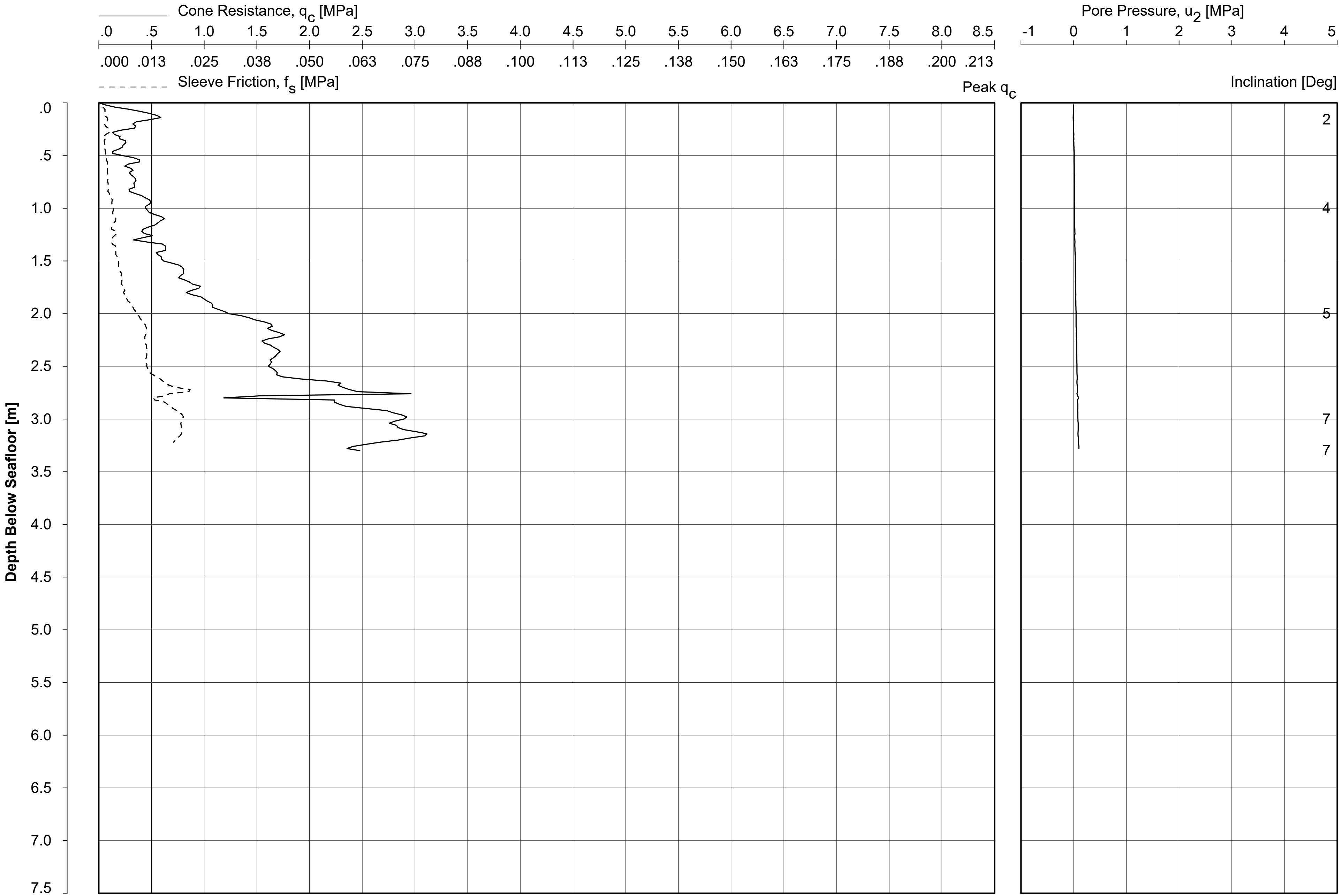
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT10

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:52



Date Of Testing : 19-Apr-2017
Water Depth [m] : 36.3
Coordinates [m] : E494625 N6021237

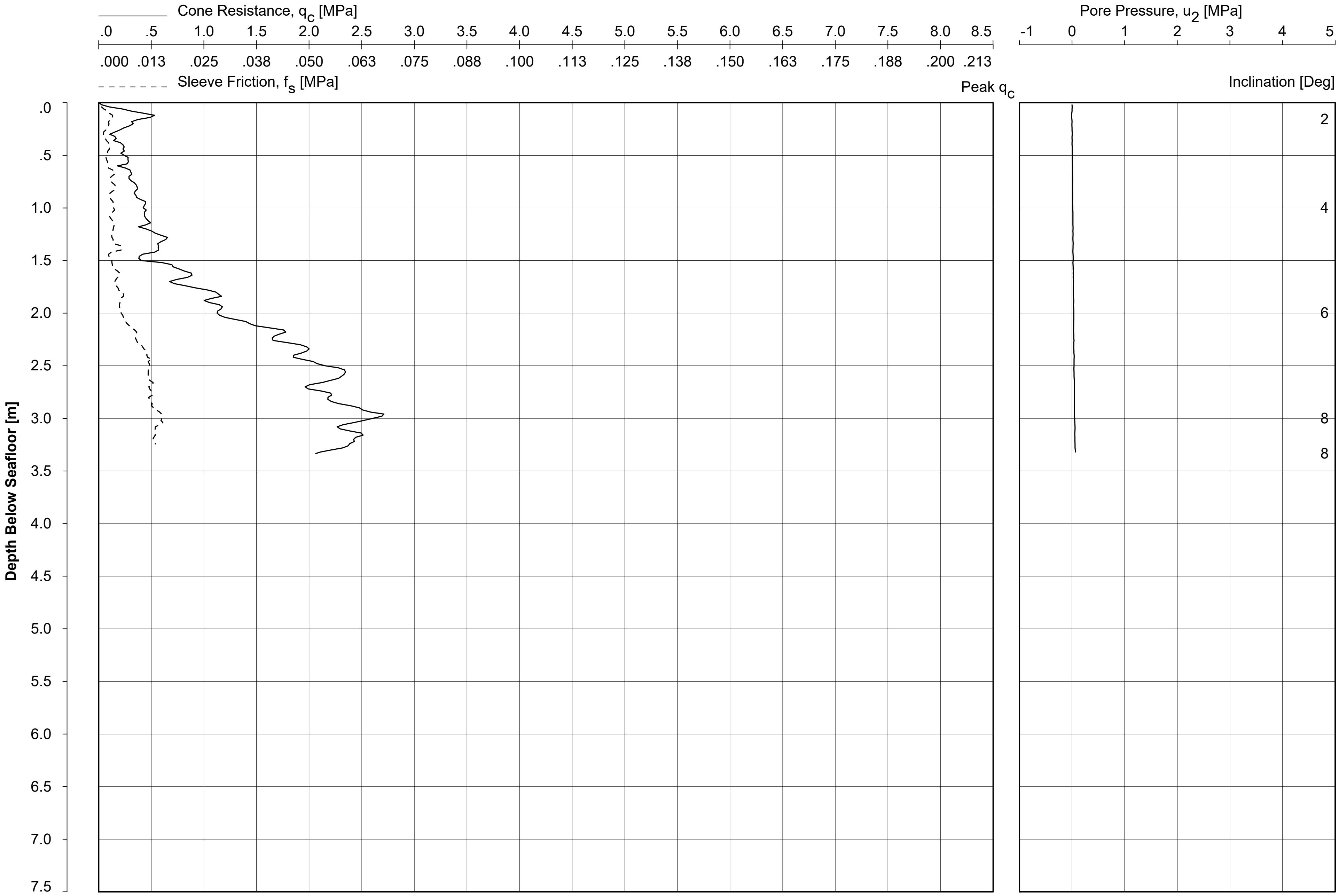
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT11

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:52



Date Of Testing : 20-Apr-2017
Water Depth [m] : 38.2
Coordinates [m] : E495273 N6020475

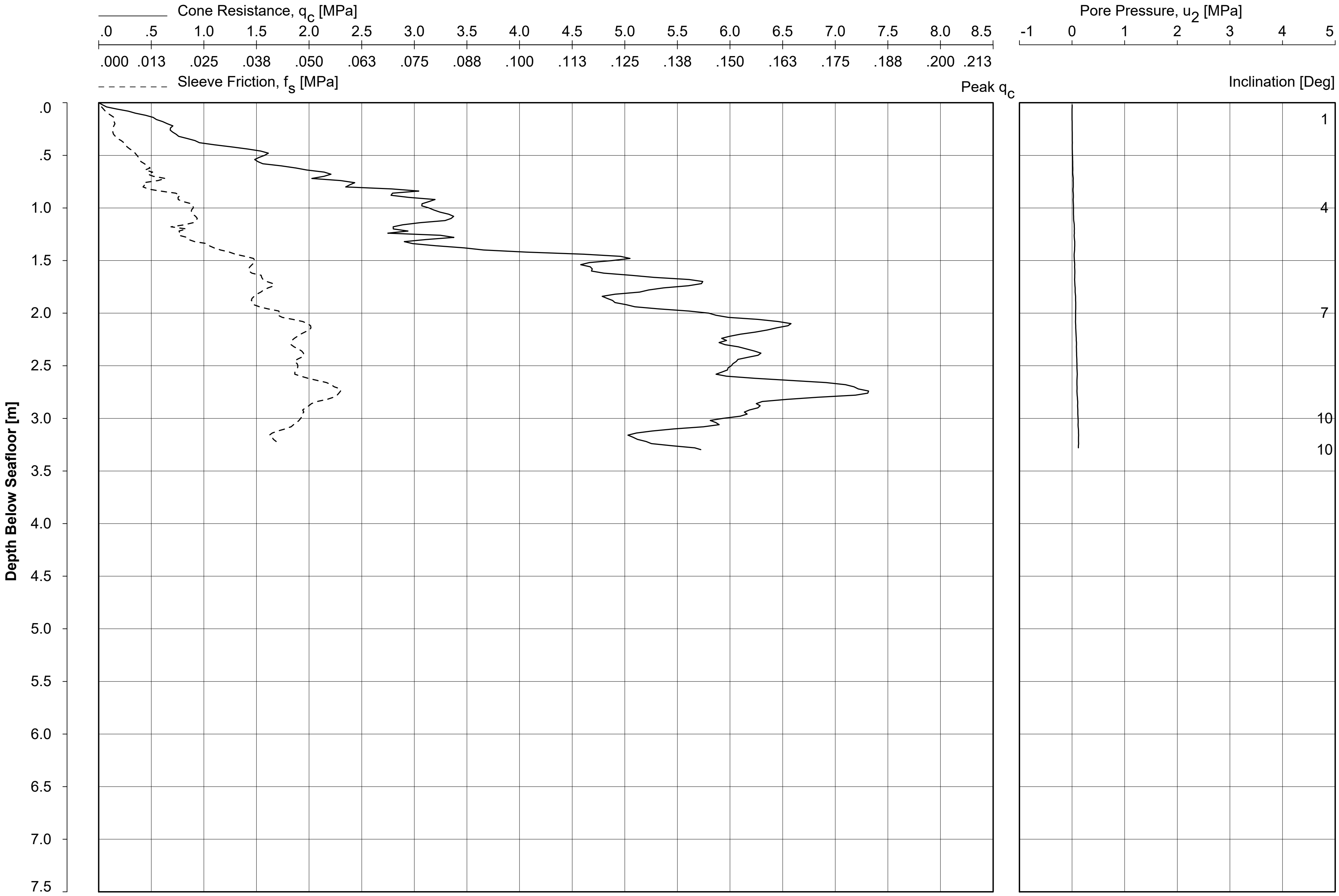
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT12

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:52



Date Of Testing : 19-Apr-2017
Water Depth [m] : 28.8
Coordinates [m] : E488571 N6027954

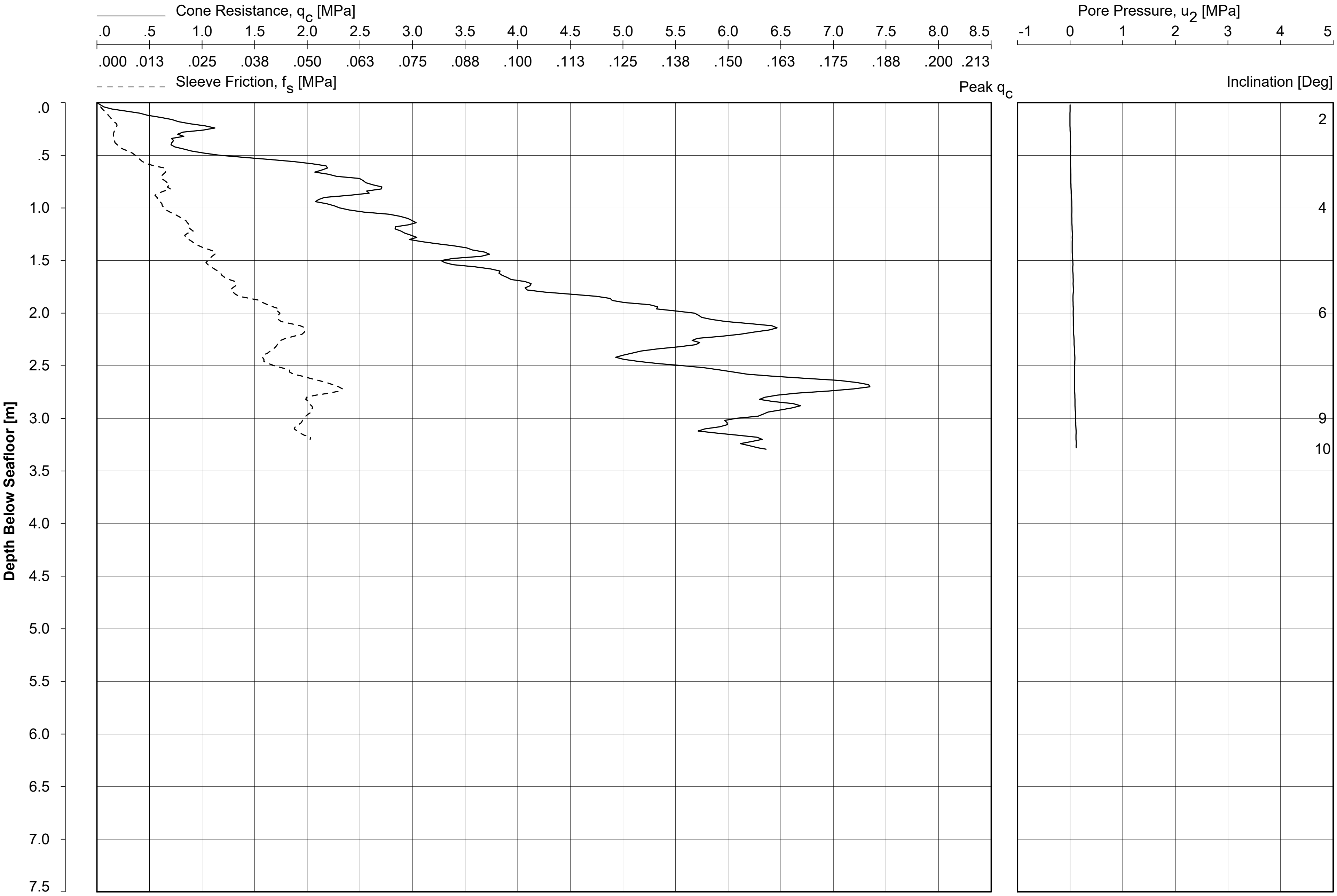
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT13

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:52



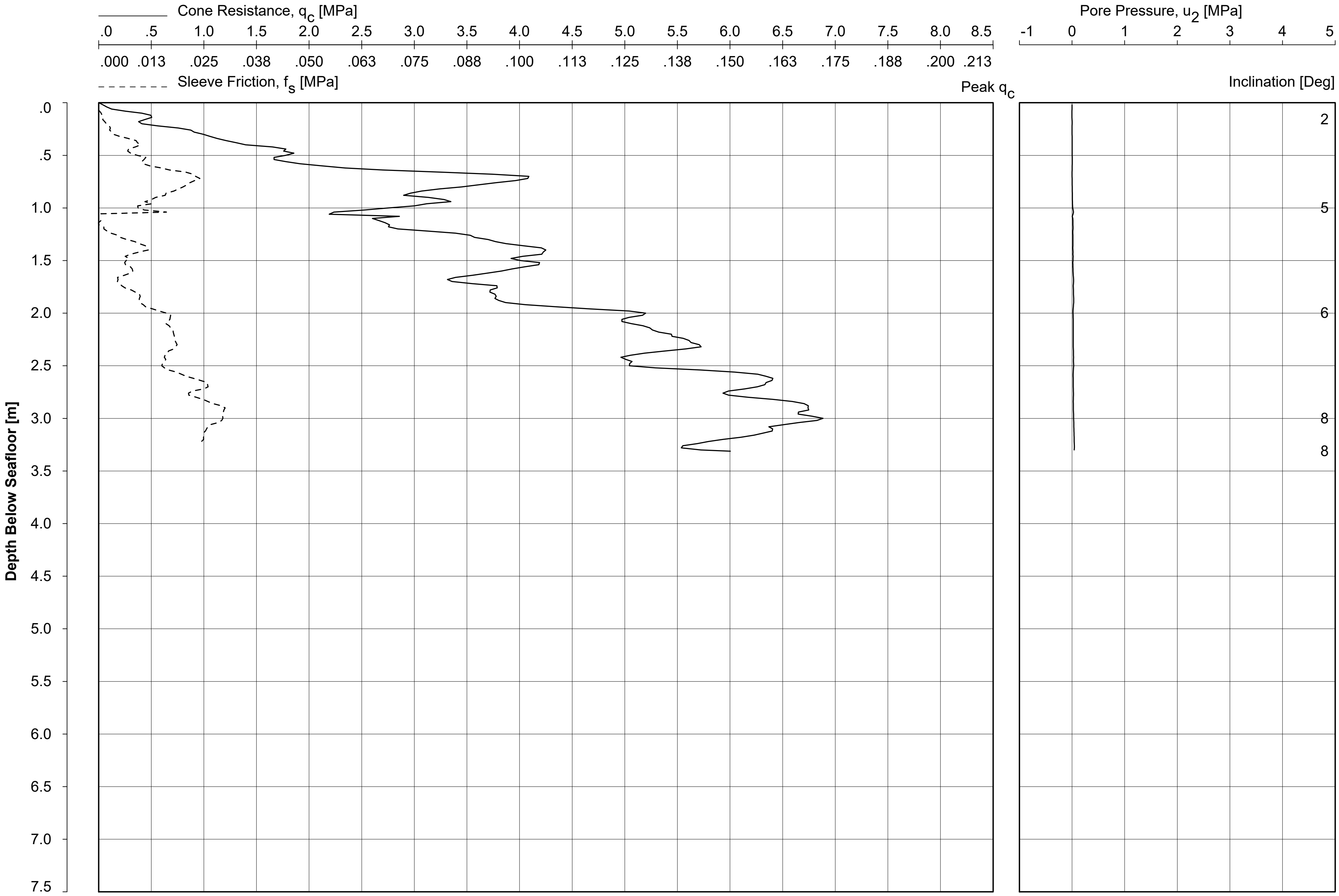
Date Of Testing	: 19-Apr-2017	Probe Type	: CP5-CF35PB25SN2
Water Depth [m]	: 29.1	Cone Base Area [mm^2]	: 503
Coordinates [m]	: E488980 N6027041		

CONE PENETRATION TEST

CPT14

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:52



Date Of Testing : 20-Apr-2017
Water Depth [m] : 29.4
Coordinates [m] : E489403 N6026101

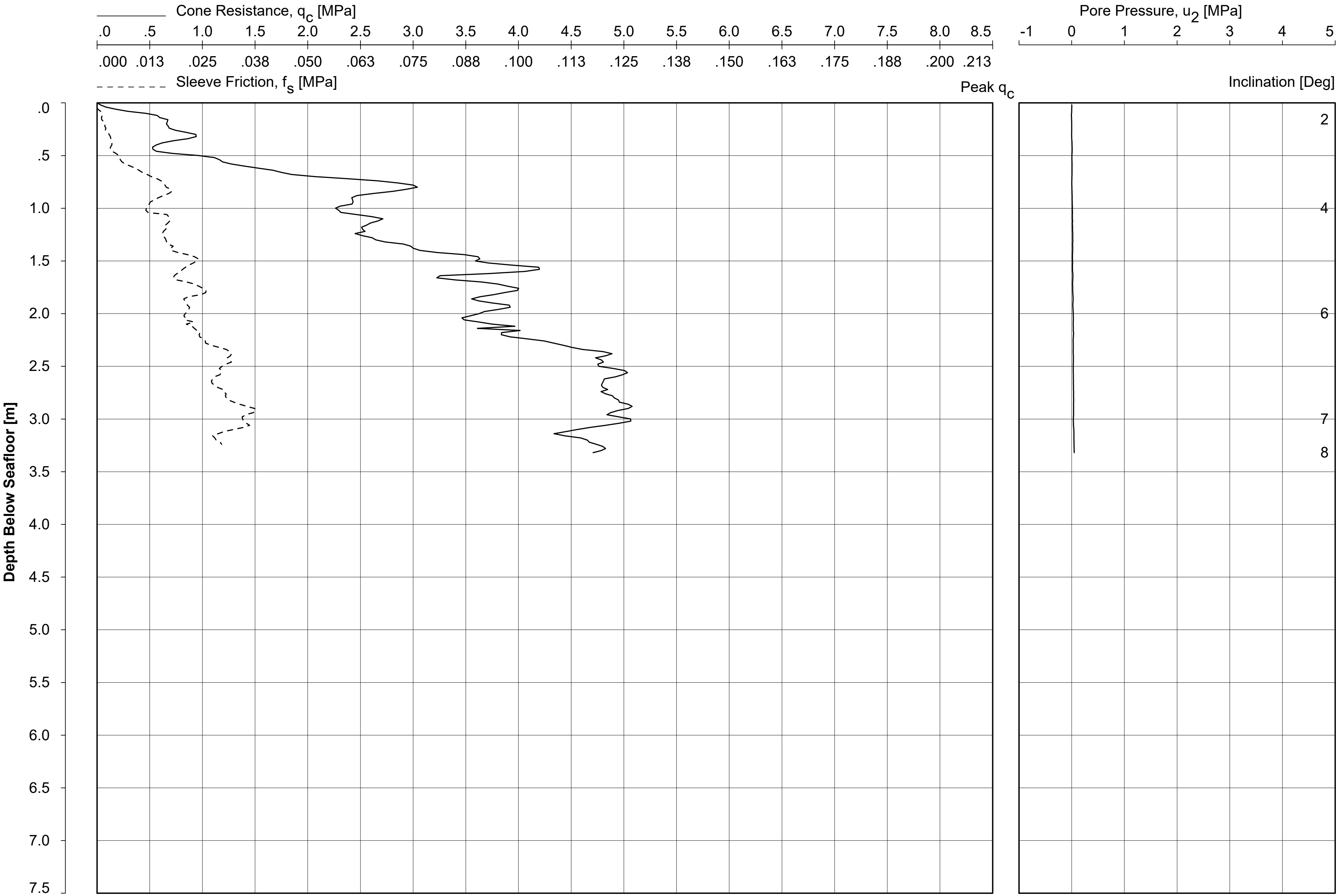
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT15

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:53



Date Of Testing : 20-Apr-2017
Water Depth [m] : 29.8
Coordinates [m] : E489801 N6025217

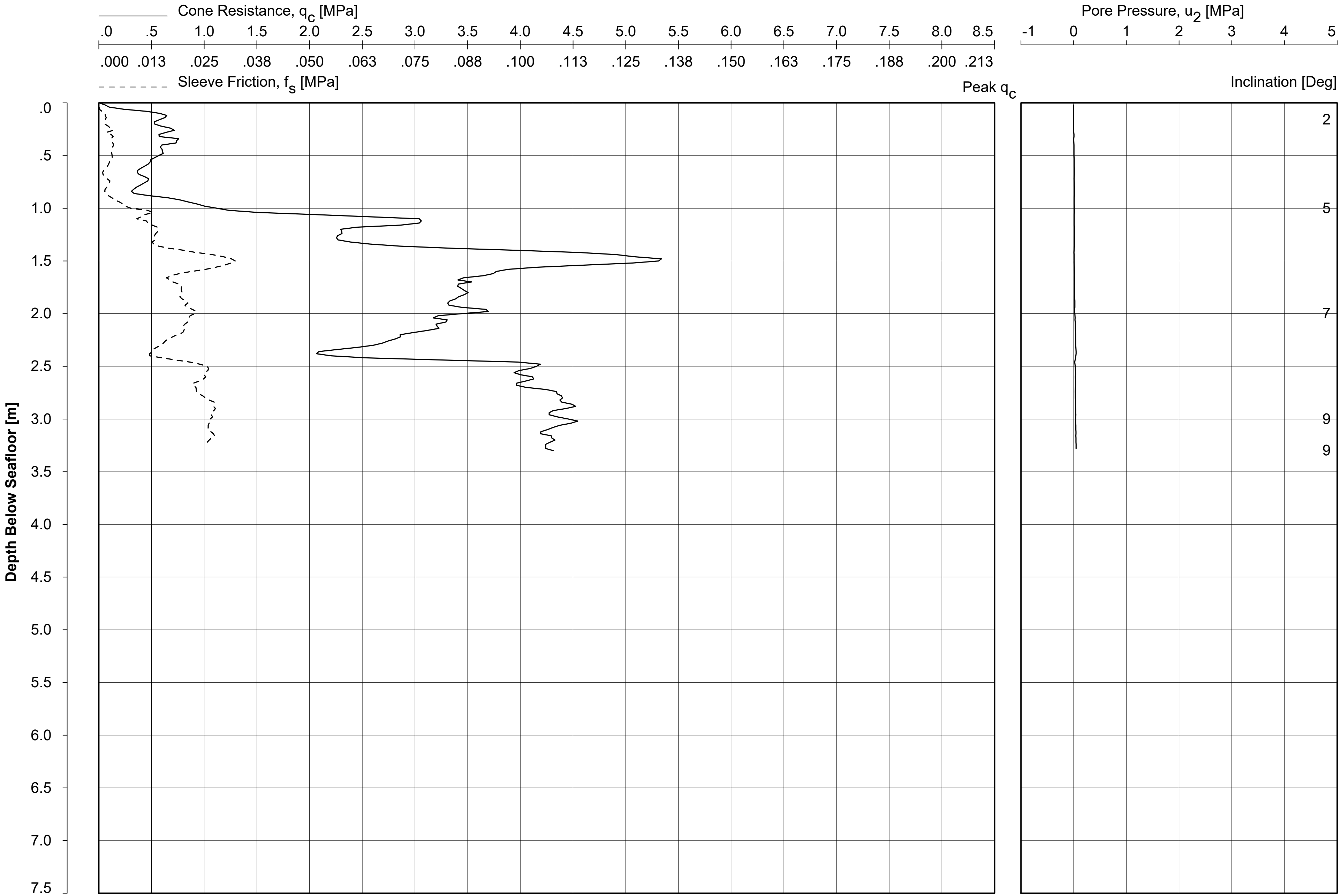
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT16

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:53



Date Of Testing : 20-Apr-2017
Water Depth [m] : 30.3
Coordinates [m] : E490211 N6024306

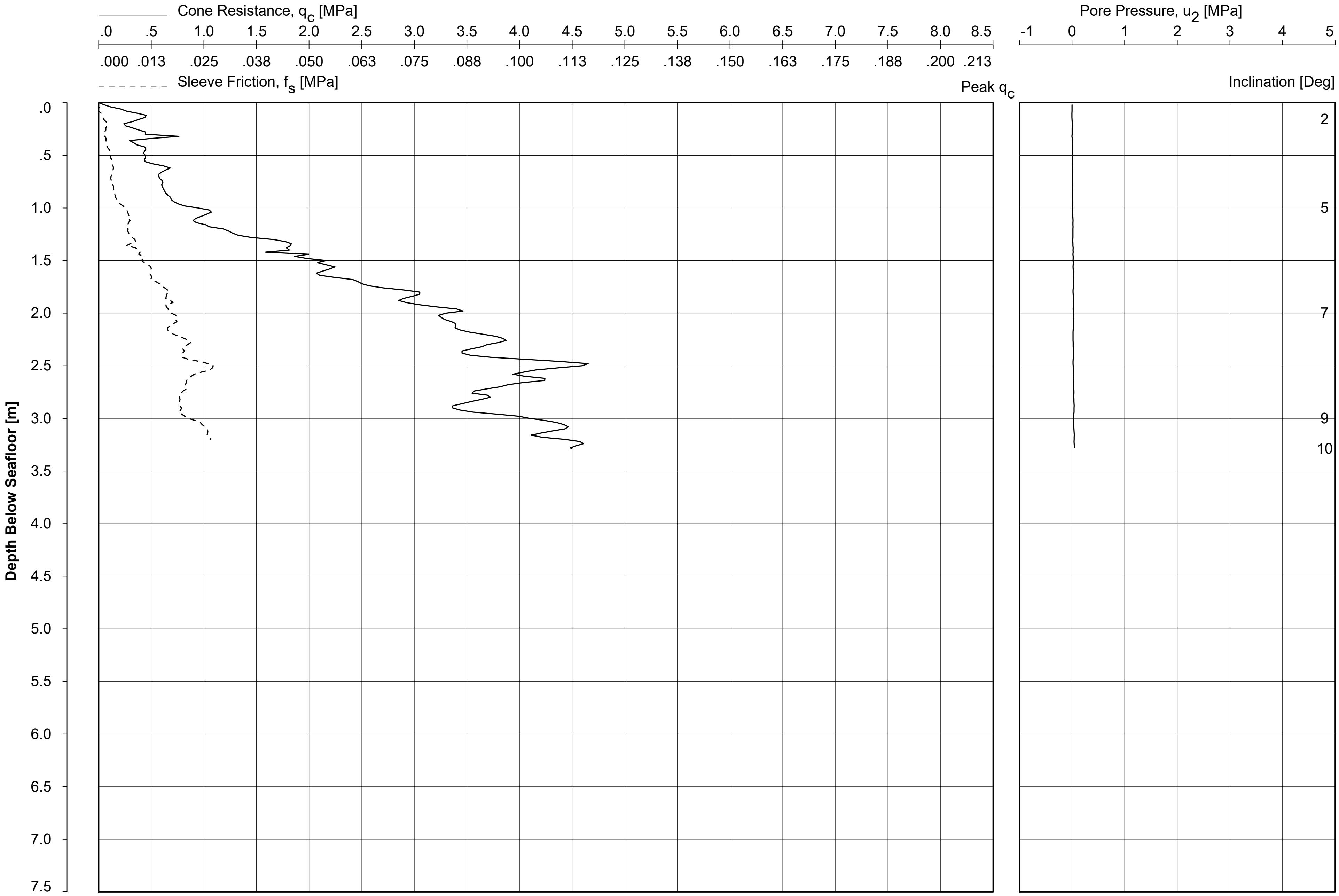
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT17

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:53



Date Of Testing : 20-Apr-2017
Water Depth [m] : 30.8
Coordinates [m] : E490619 N6023393

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

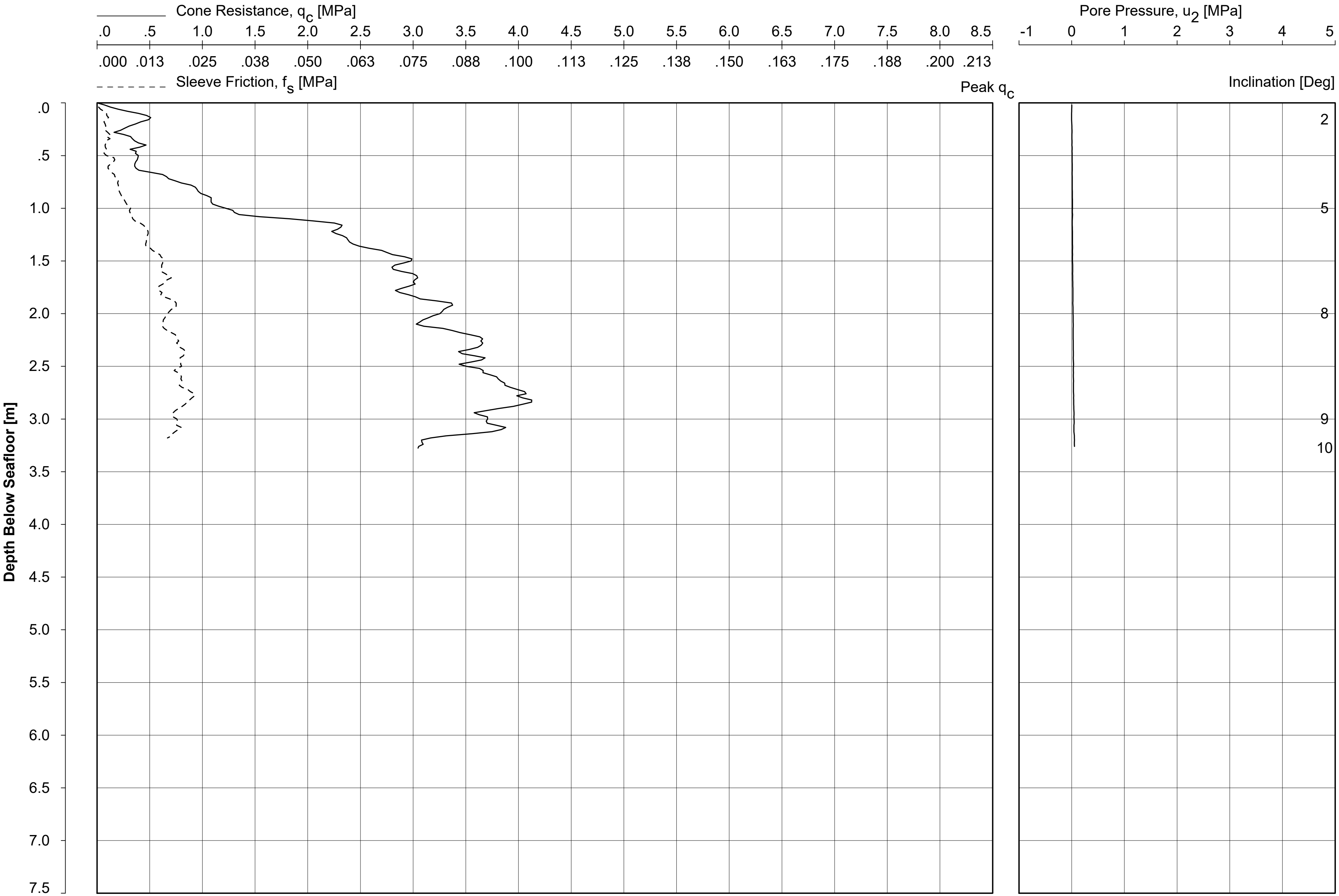
CPT18

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

FUGRO

Fugro Report No. GH210-R3 (1)

UNIPLOT 05.33.nl / Seacalf qc 05MPa.ucf / 2017-05-11 14:59:53



Date Of Testing : 20-Apr-2017
Water Depth [m] : 31.4
Coordinates [m] : E491029 N6022481

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

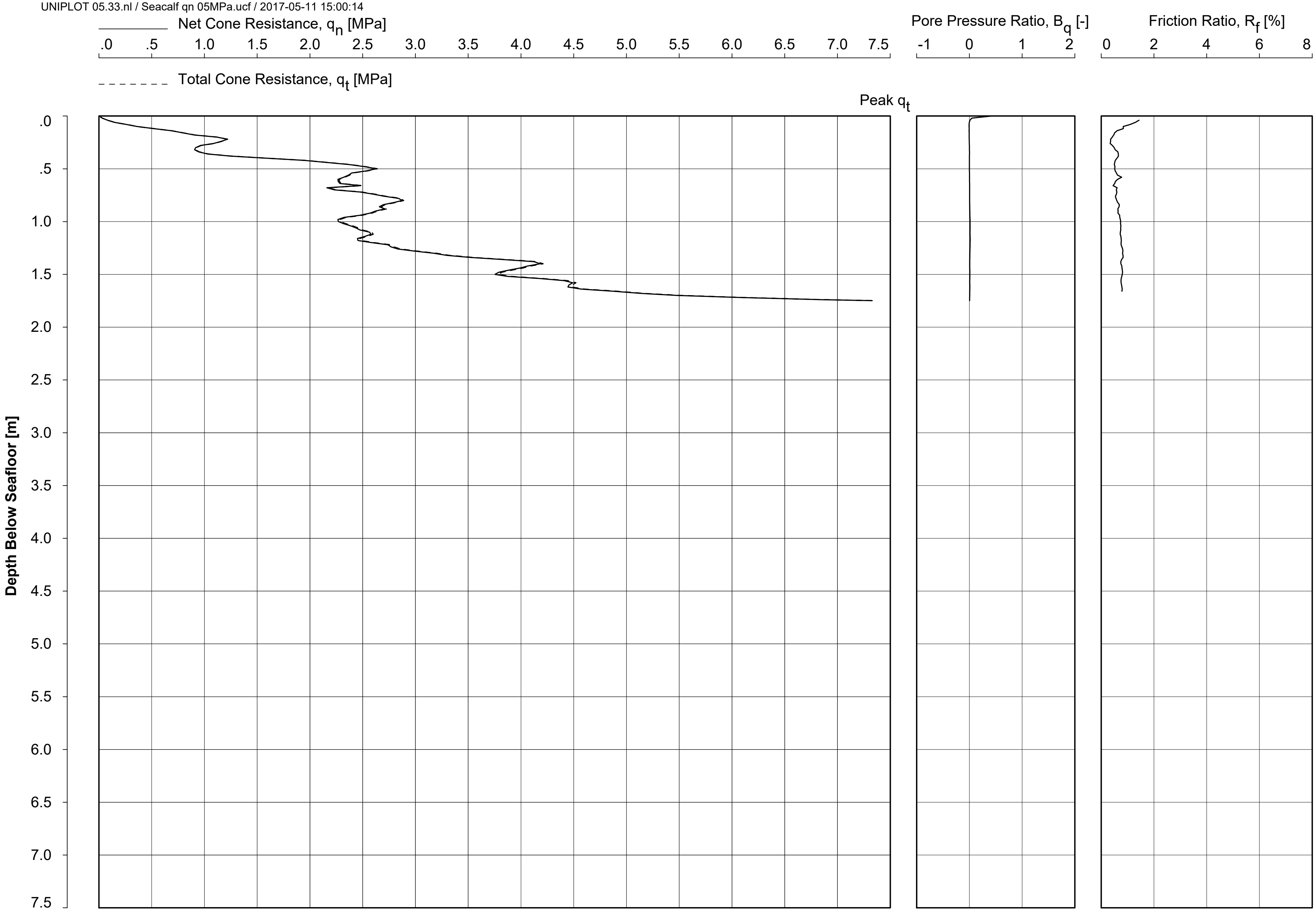
CONE PENETRATION TEST

CPT19

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

FUGRO

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:14



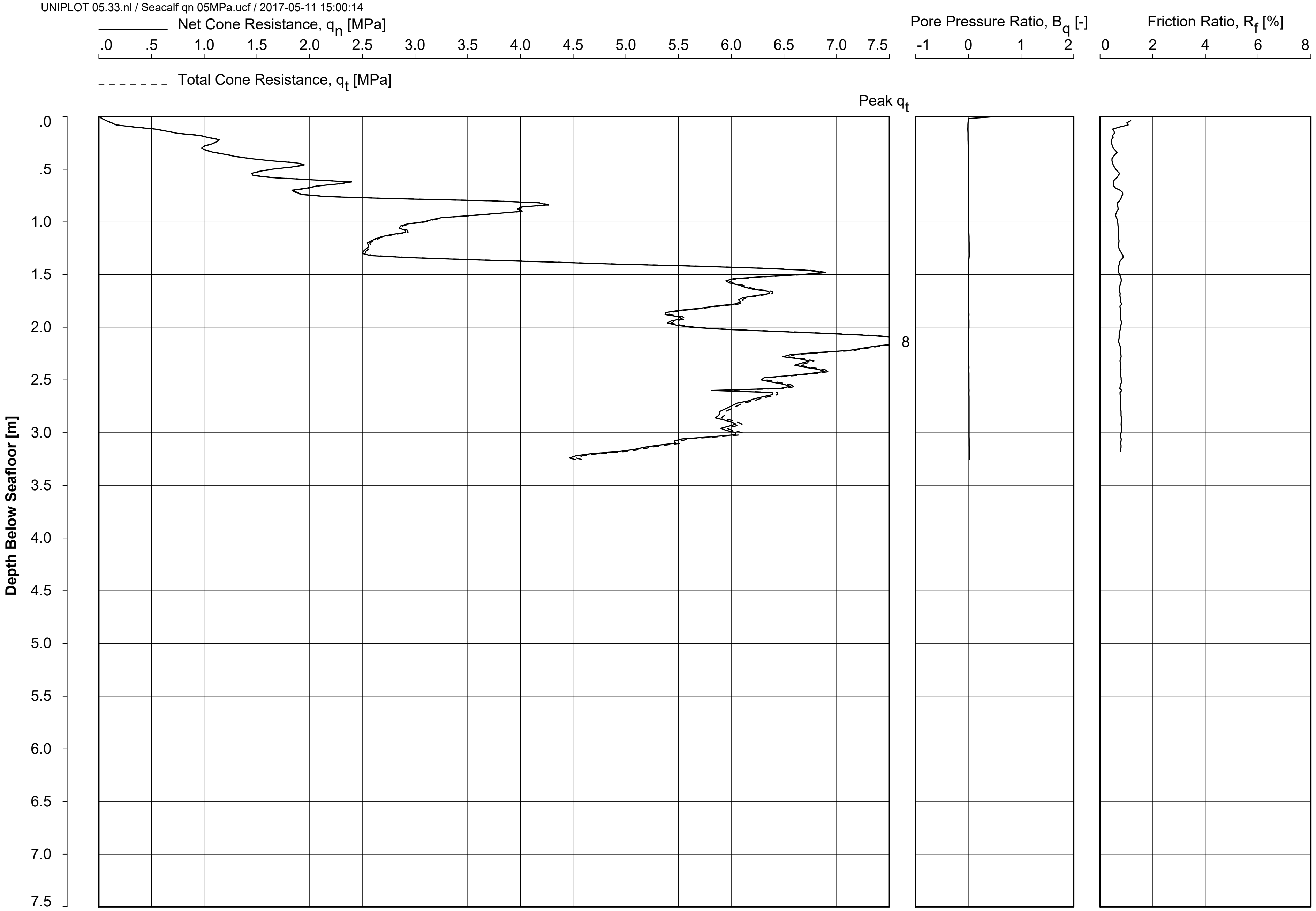
Date Of Testing : 19-Apr-2017
Water Depth [m] : 28.6
Coordinates [m] : E488162 N6028866

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT01

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA



Date Of Testing : 19-Apr-2017
Water Depth [m] : 28.9
Coordinates [m] : E488807 N6028105

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

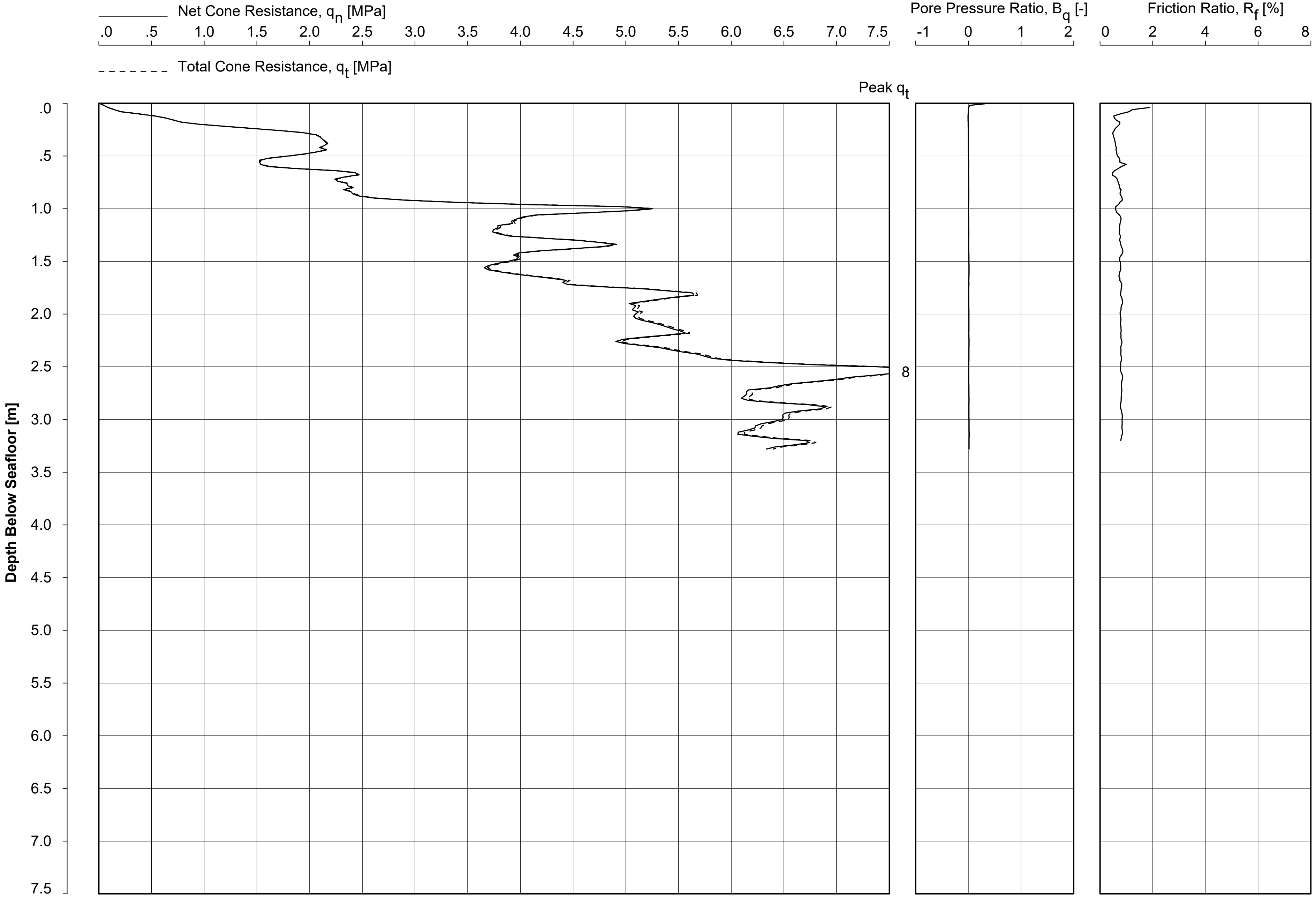
CONE PENETRATION TEST

CPT02

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

FUGRO

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:14



Date Of Testing : 19-Apr-2017
Water Depth [m] : 29.3
Coordinates [m] : E489454 N6027342

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

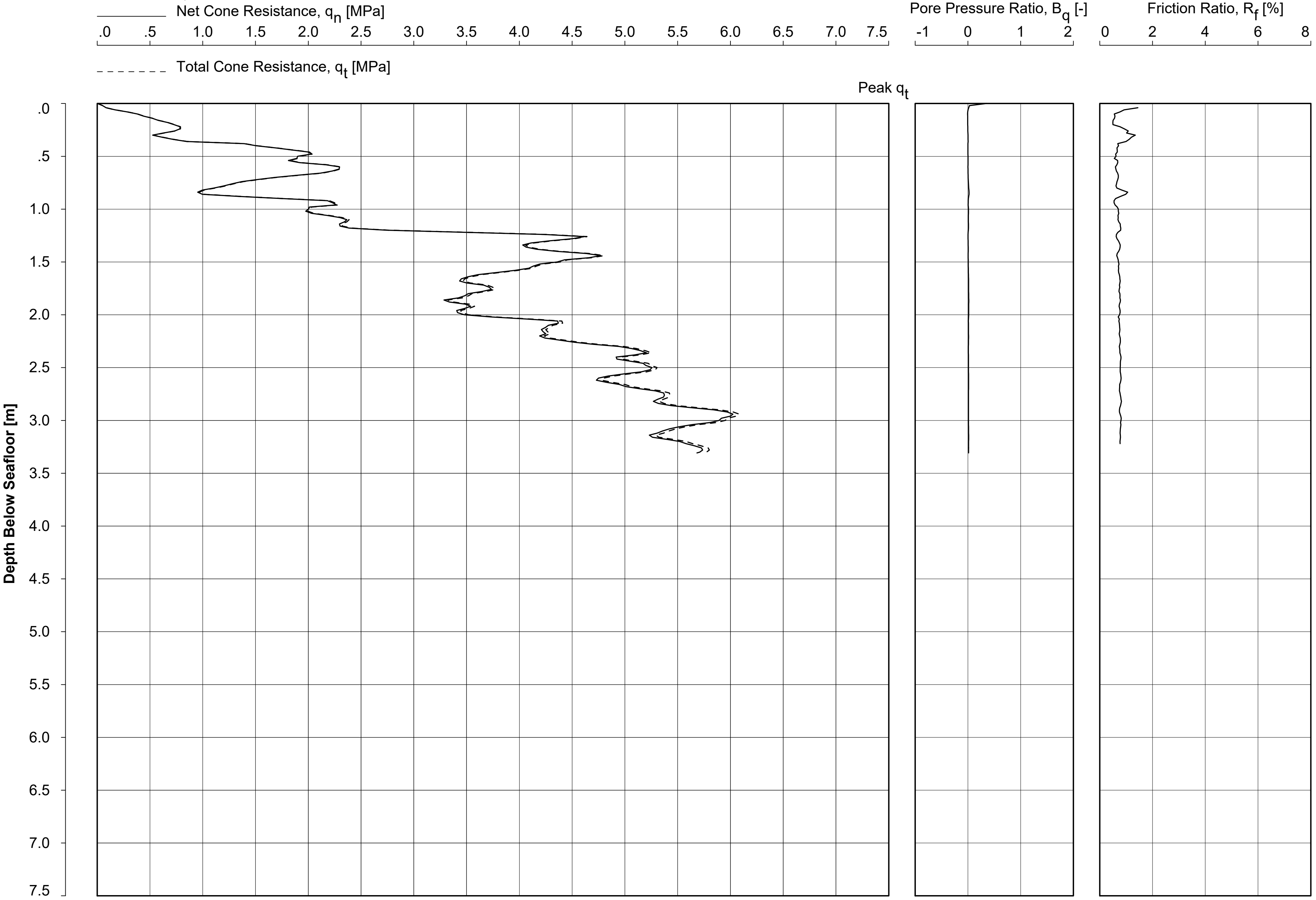
CONE PENETRATION TEST

CPT03

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

FUGRO

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:14



Date Of Testing : 19-Apr-2017
Water Depth [m] : 29.6
Coordinates [m] : E490101 N6026576

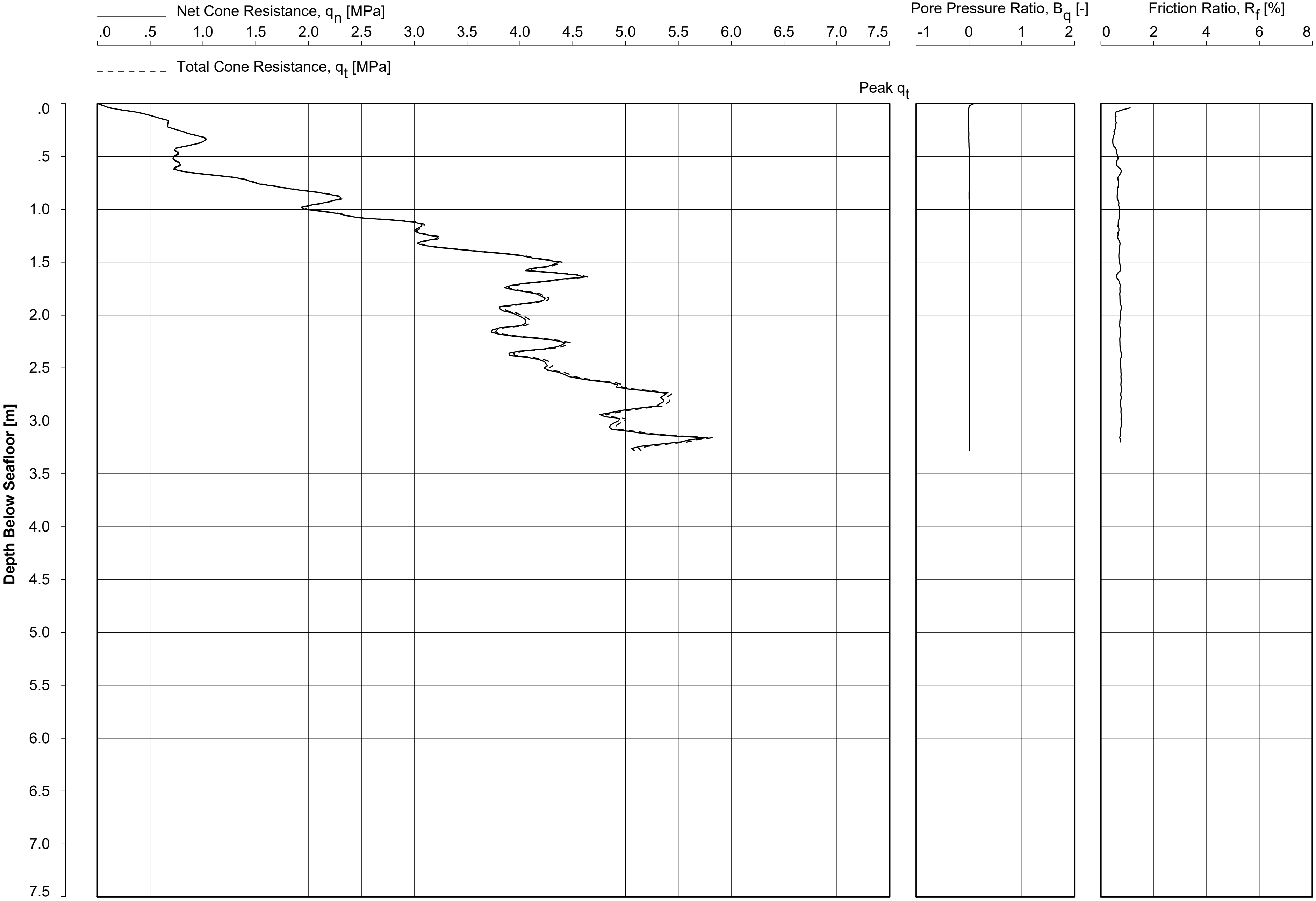
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT04

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:14



Date Of Testing : 19-Apr-2017
Water Depth [m] : 30.0
Coordinates [m] : E490747 N6025815

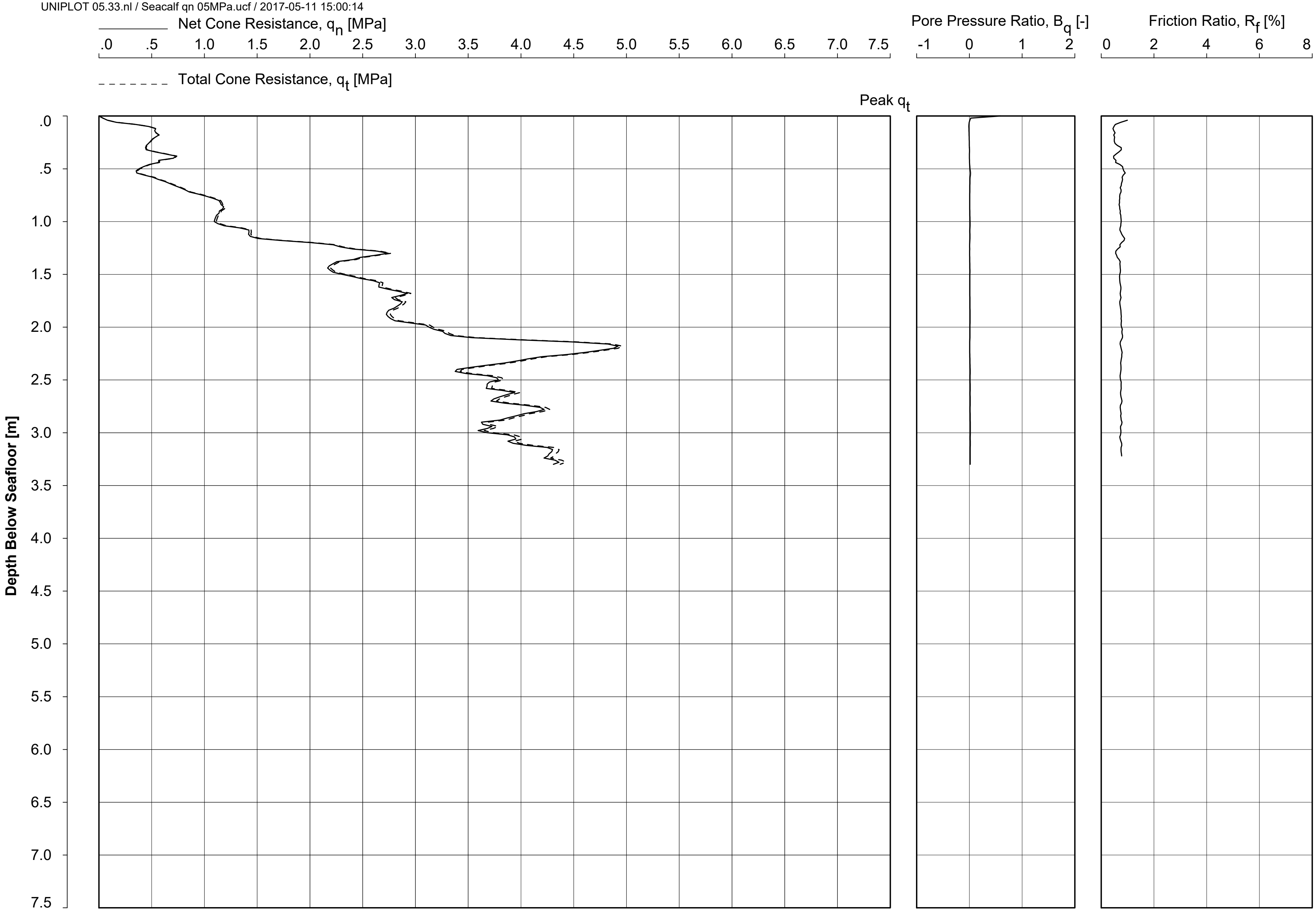
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT05

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:14



Date Of Testing : 19-Apr-2017
Water Depth [m] : 30.6
Coordinates [m] : E491394 N6025052

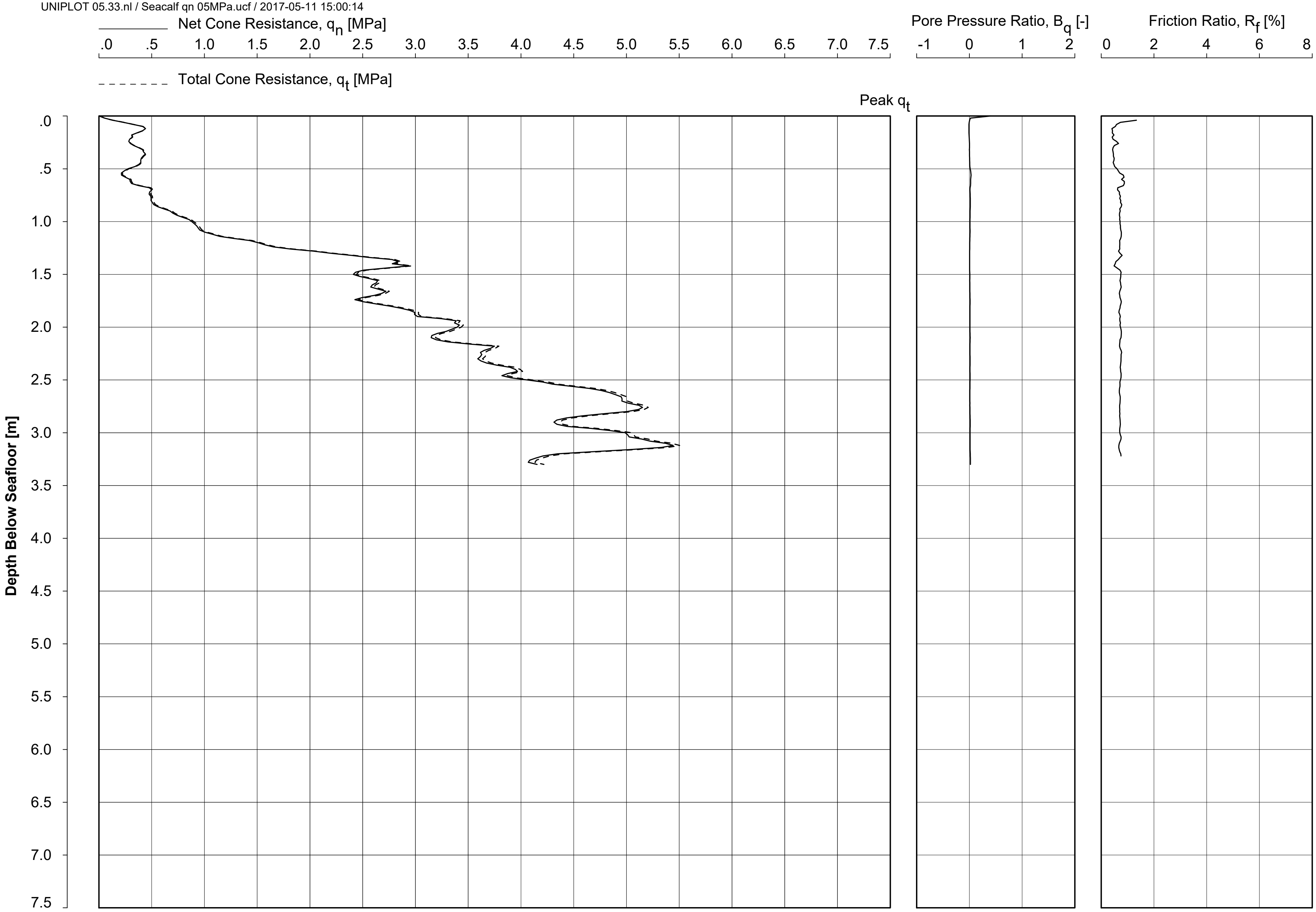
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT06

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:14



Date Of Testing : 19-Apr-2017
Water Depth [m] : 31.2
Coordinates [m] : E492040 N6024287

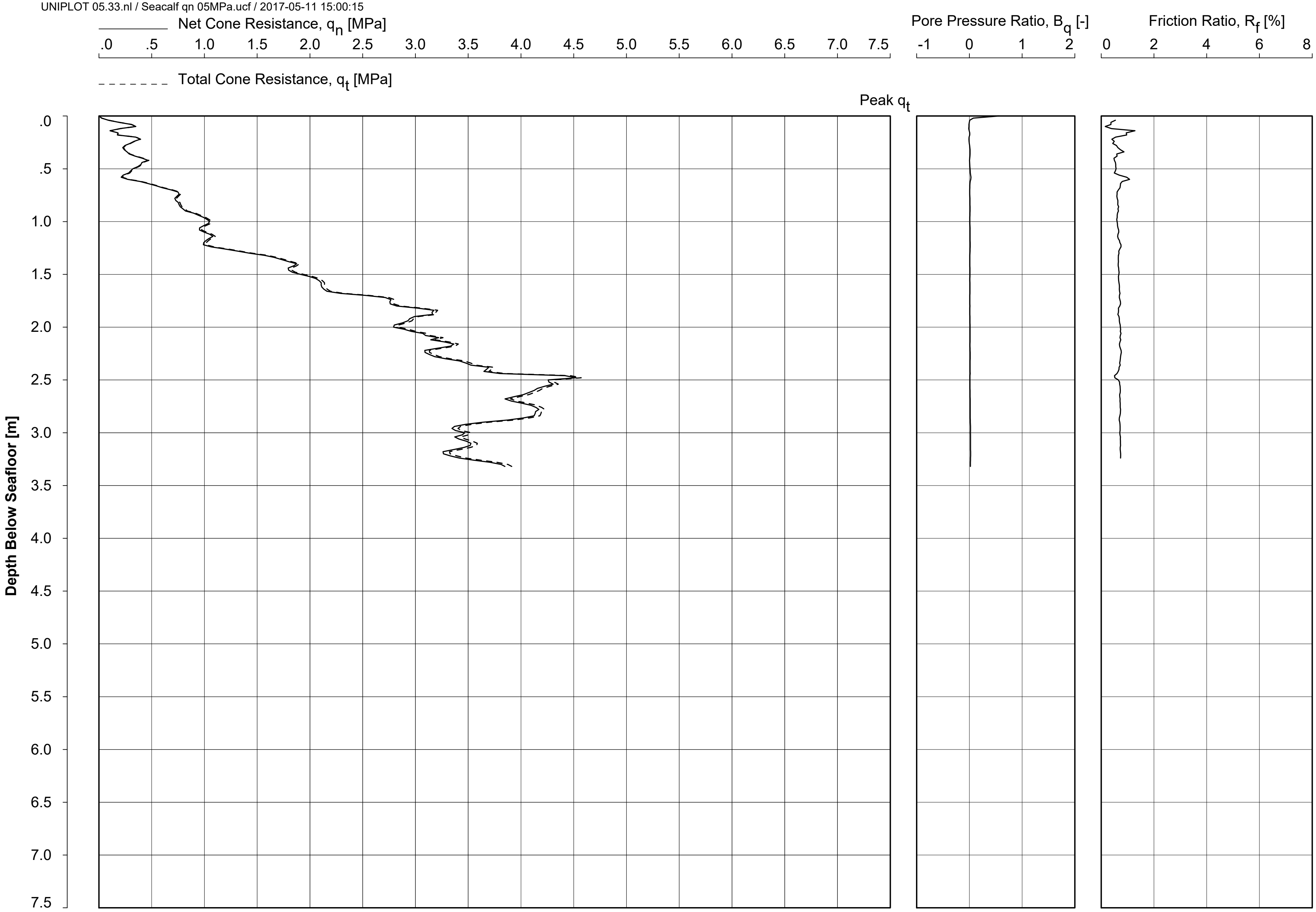
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT07

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



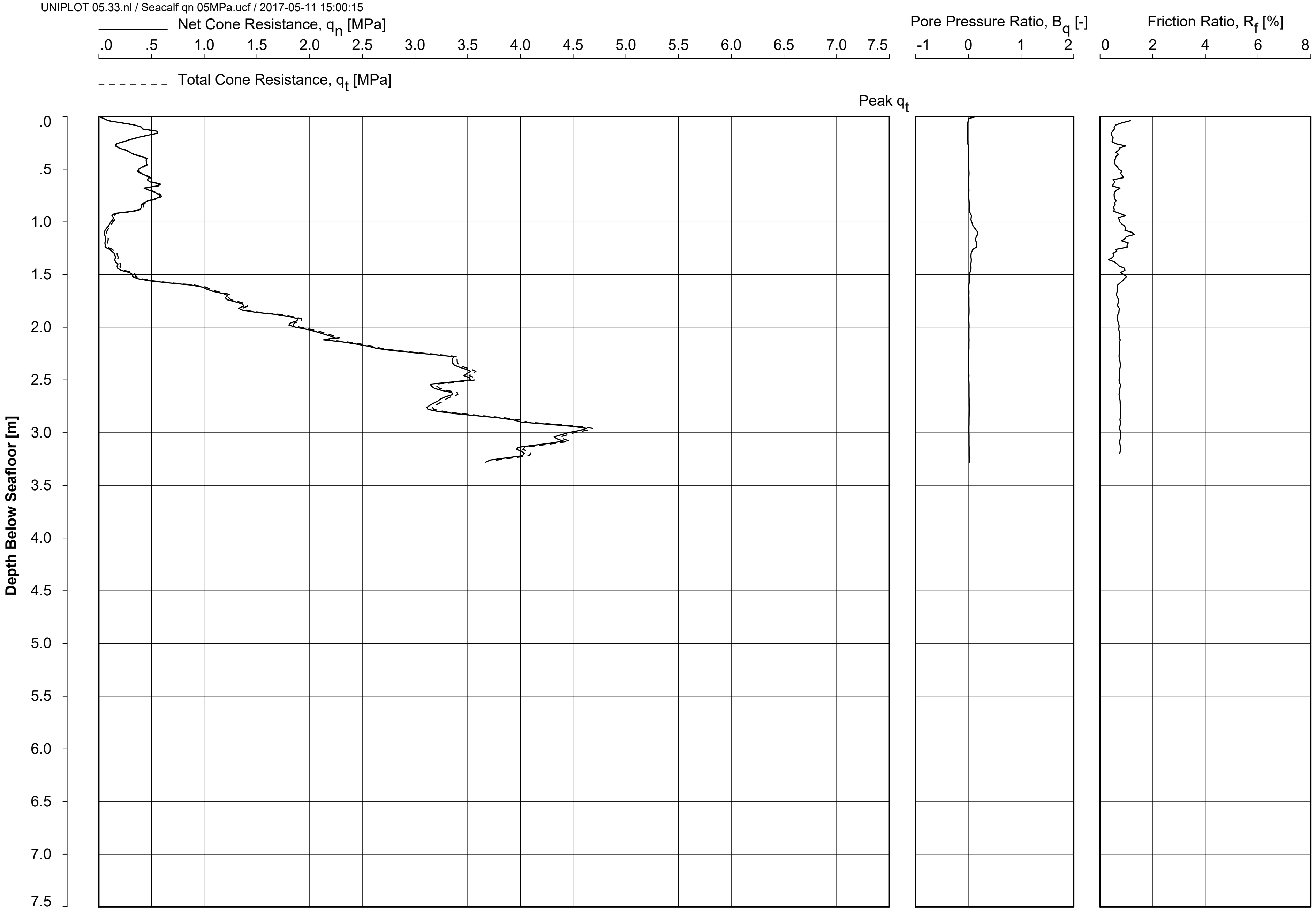
Date Of Testing : 19-Apr-2017
Water Depth [m] : 31.9
Coordinates [m] : E492687 N6023526

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT08

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA



Date Of Testing : 19-Apr-2017
Water Depth [m] : 33.1
Coordinates [m] : E493327 N6022770

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

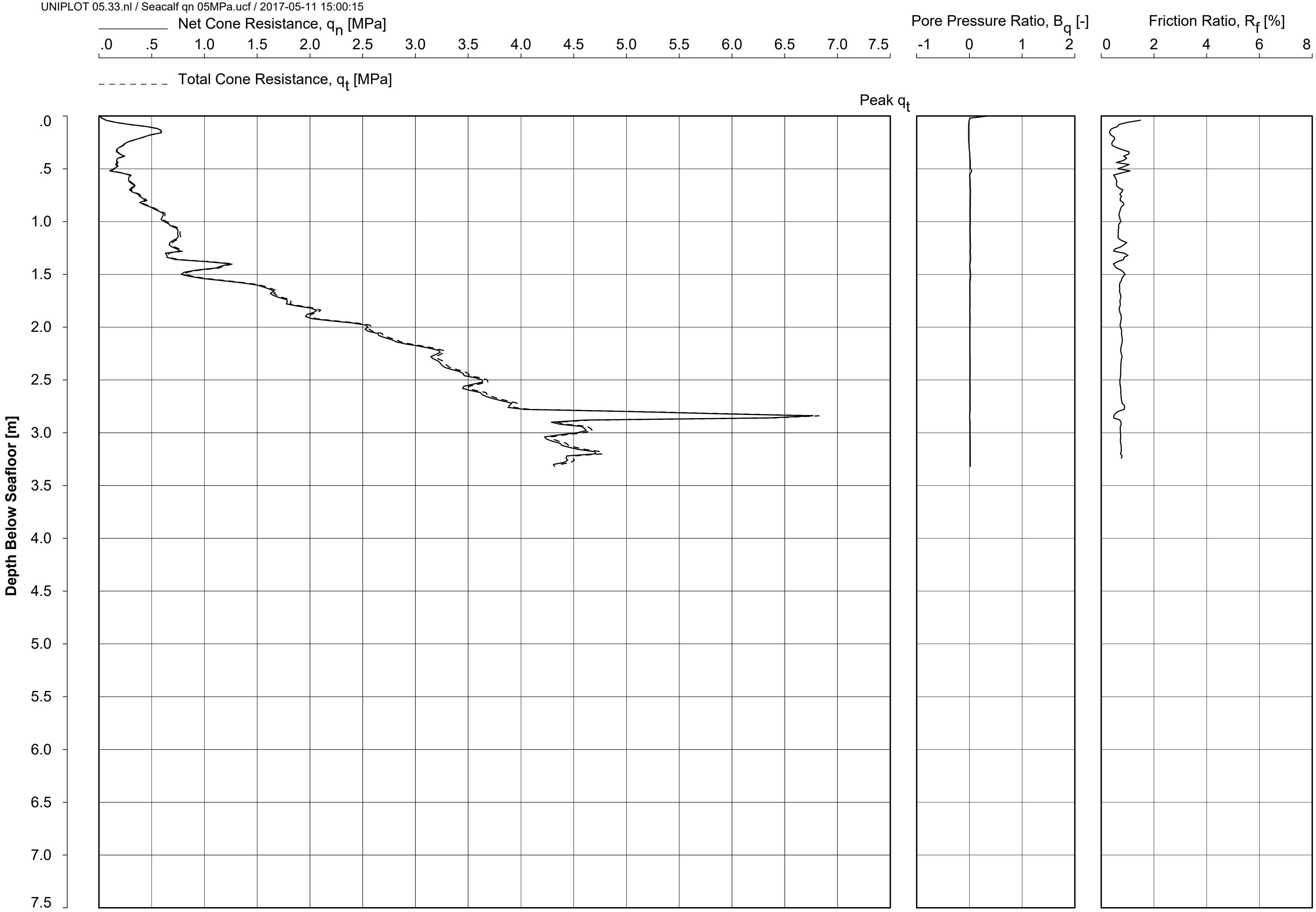
CONE PENETRATION TEST

CPT09

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

Fugro Report No. GH210-R3 (1)

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



Date Of Testing : 19-Apr-2017
Water Depth [m] : 34.6
Coordinates [m] : E493979 N6021999

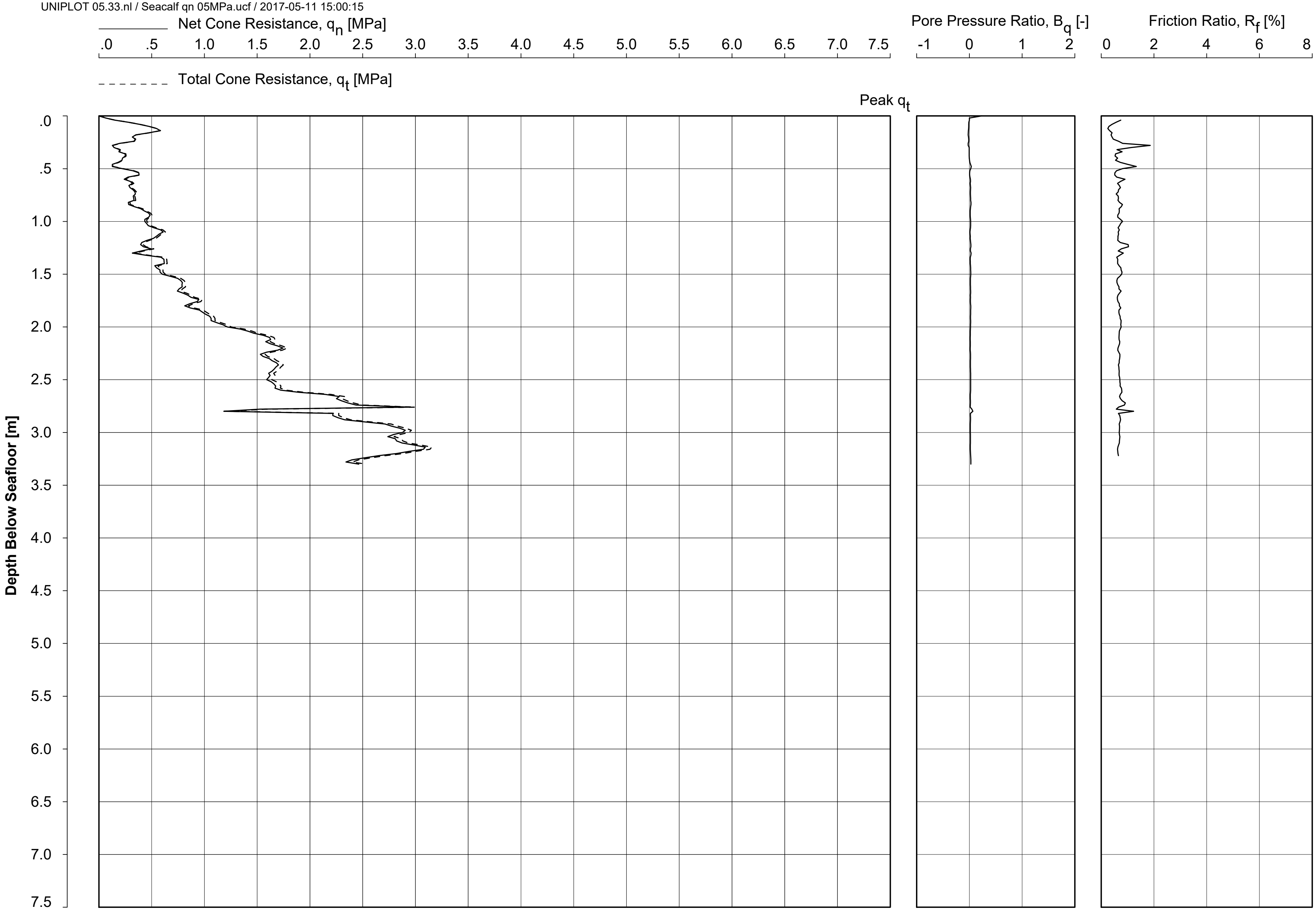
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT10

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



Date Of Testing : 19-Apr-2017
Water Depth [m] : 36.3
Coordinates [m] : E494625 N6021237

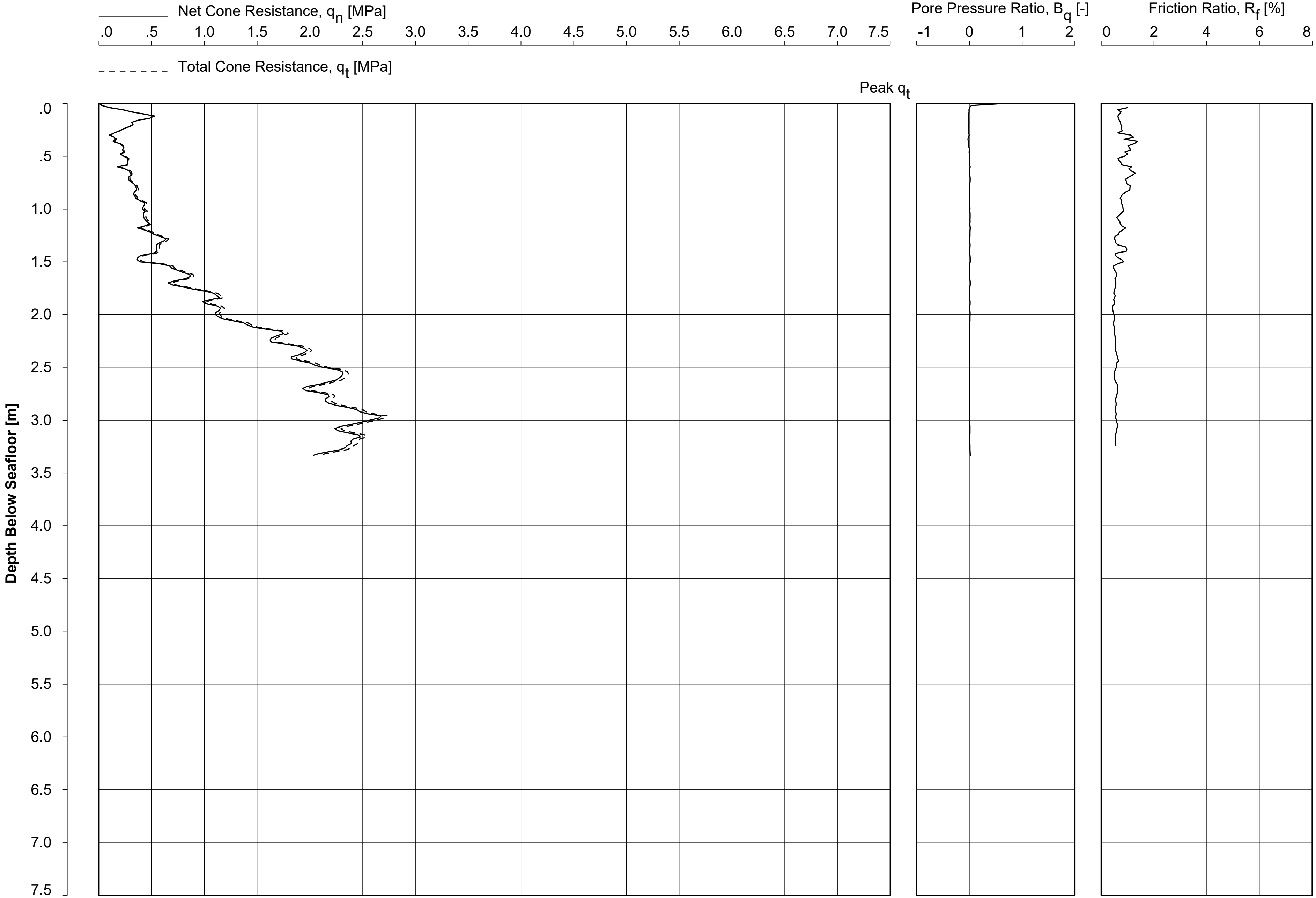
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT11

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



Date Of Testing : 20-Apr-2017
Water Depth [m] : 38.2
Coordinates [m] : E495273 N6020475

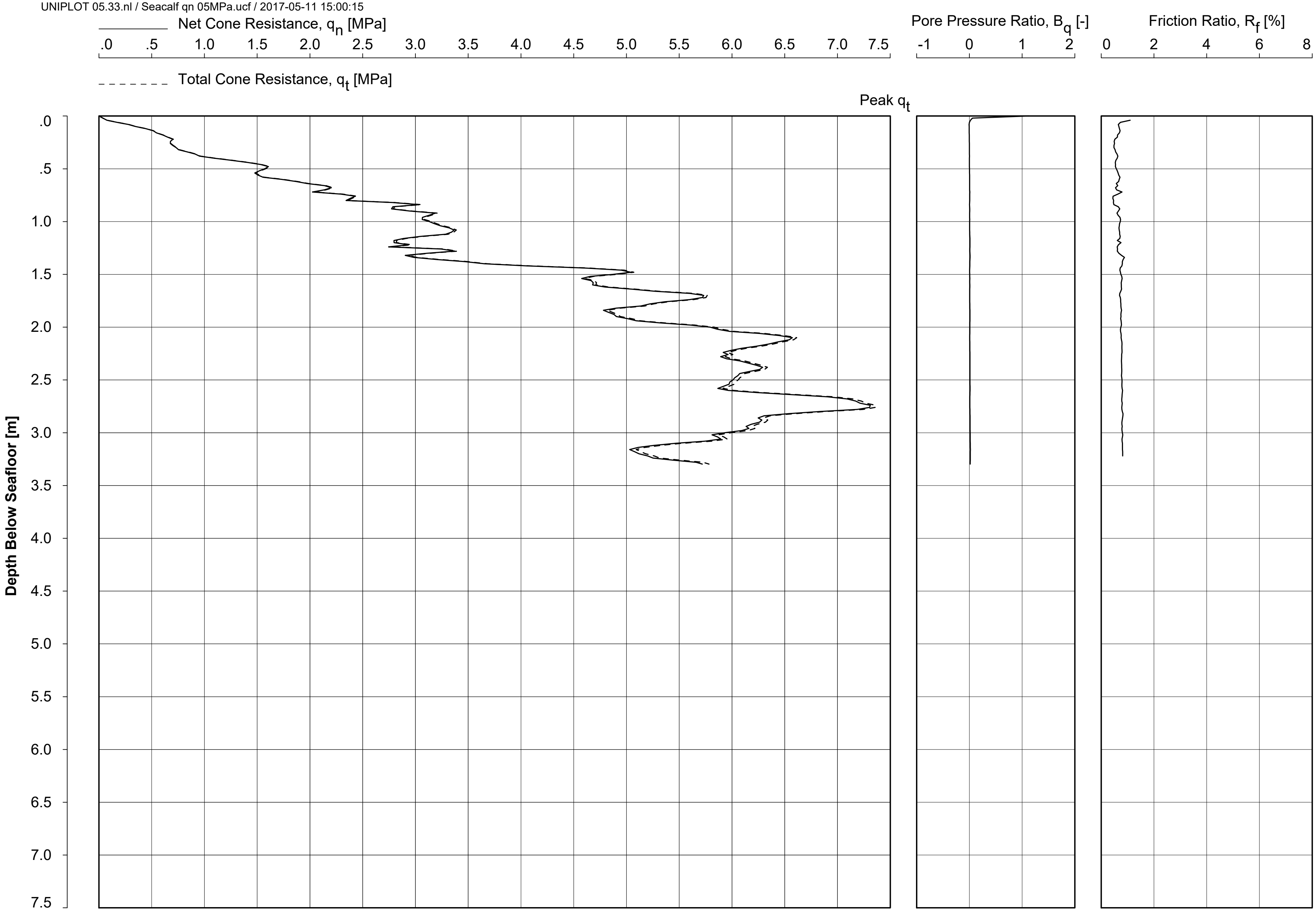
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT12

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



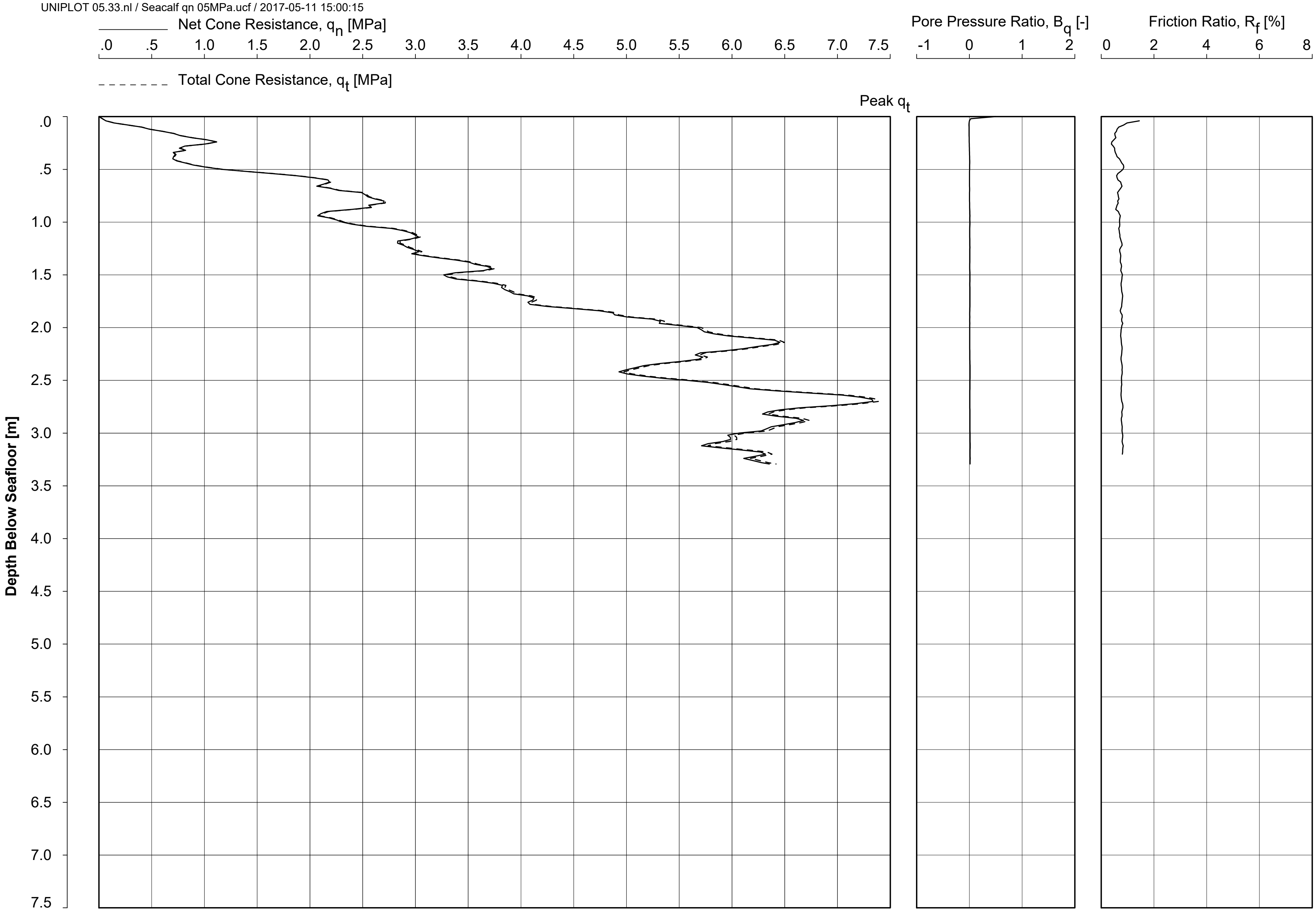
Date Of Testing : 19-Apr-2017
Water Depth [m] : 28.8
Coordinates [m] : E488571 N6027954

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT13

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA



Date Of Testing : 19-Apr-2017
Water Depth [m] : 29.1
Coordinates [m] : E488980 N6027041

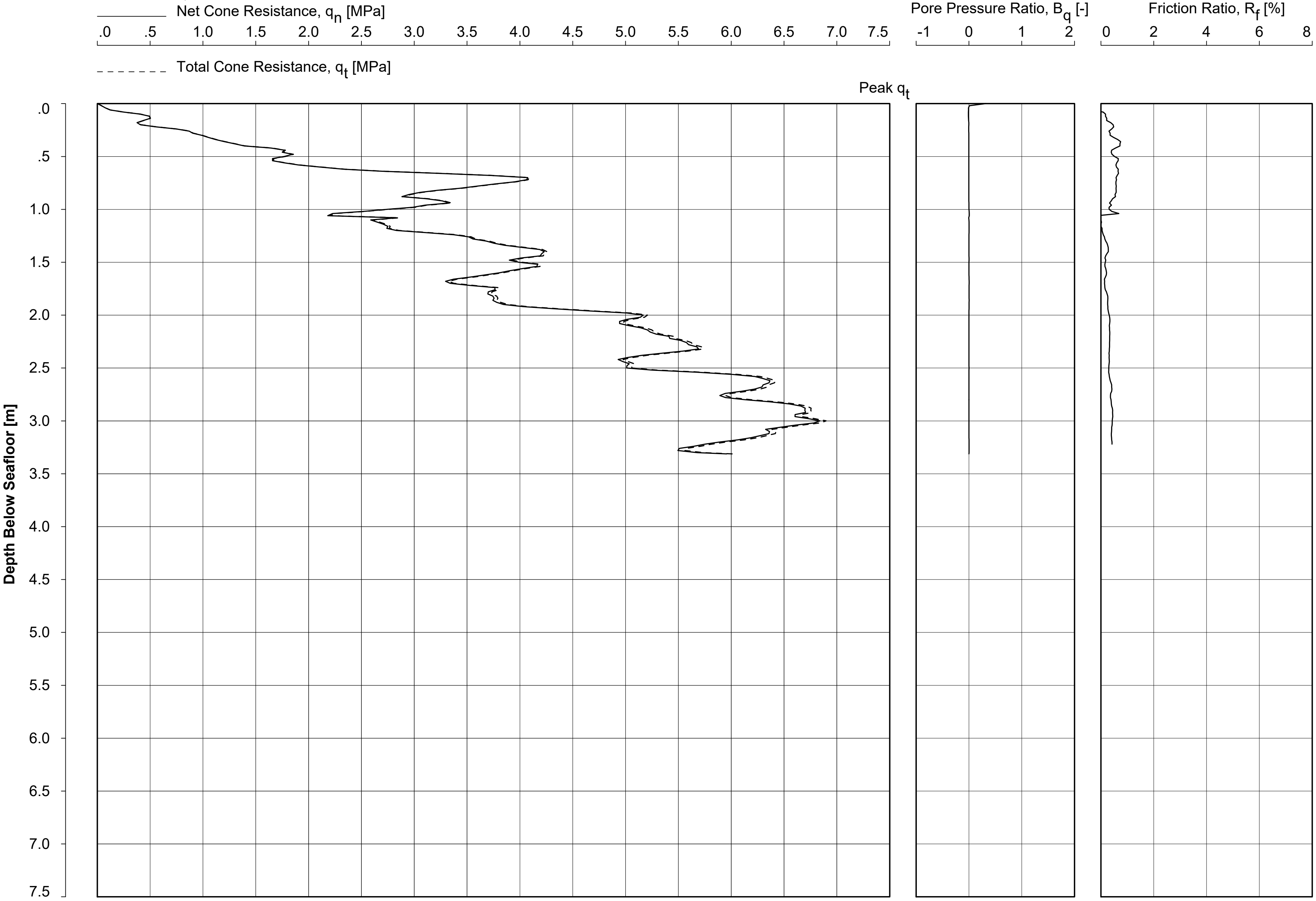
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT14

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



Date Of Testing : 20-Apr-2017
Water Depth [m] : 29.4
Coordinates [m] : E489403 N6026101

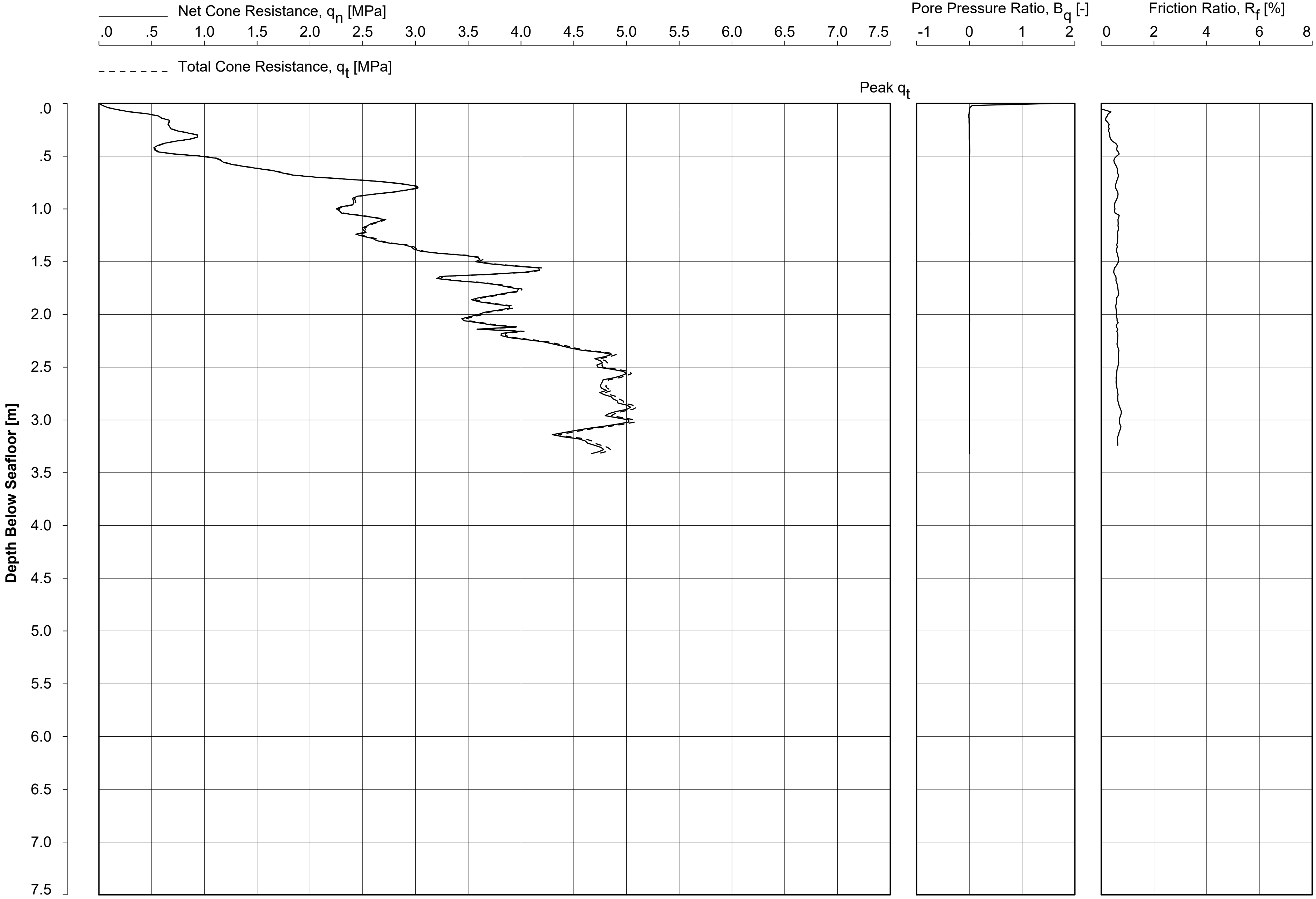
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT15

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



Date Of Testing : 20-Apr-2017
Water Depth [m] : 29.8
Coordinates [m] : E489801 N6025217

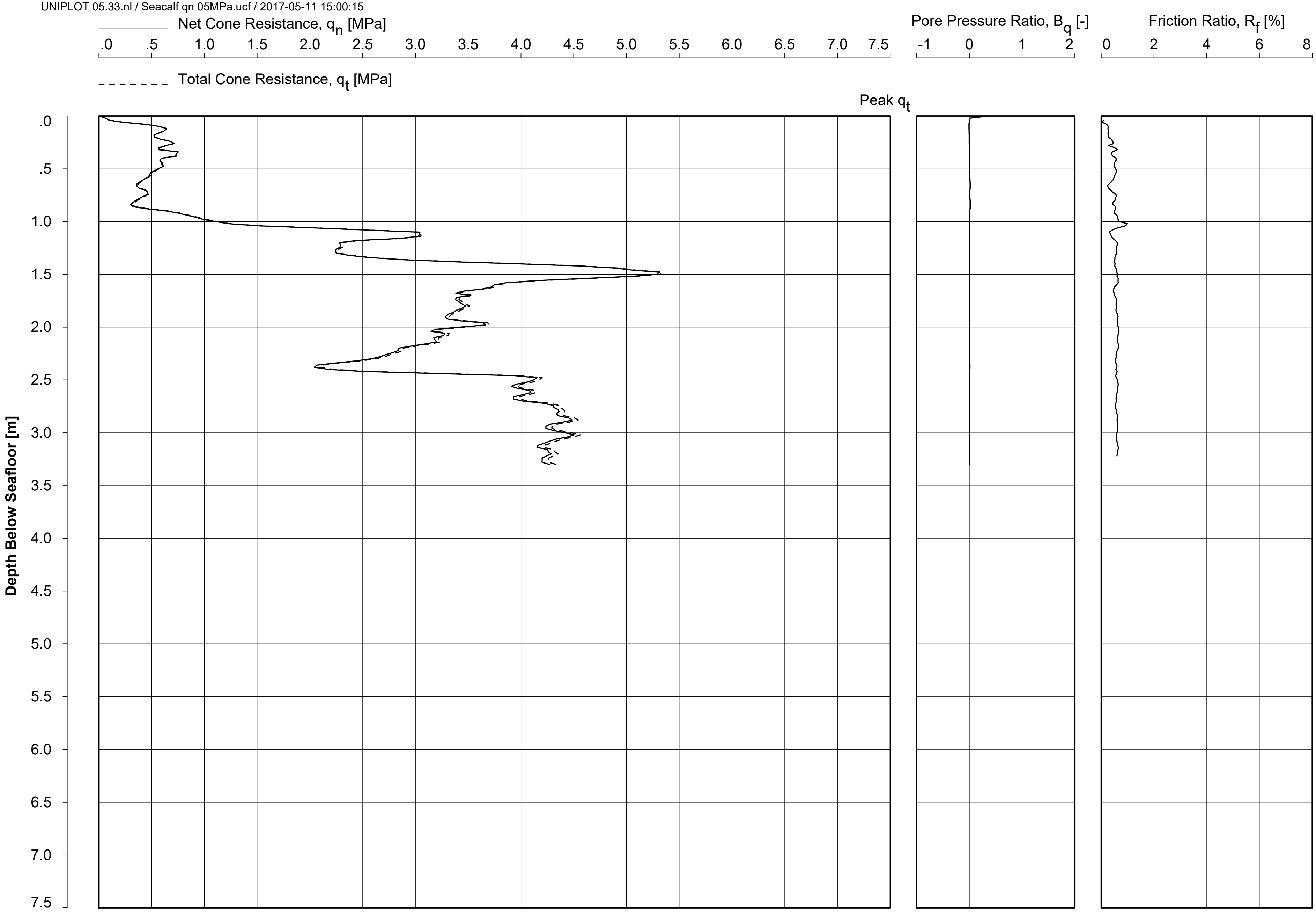
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT16

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



Date Of Testing : 20-Apr-2017
Water Depth [m] : 30.3
Coordinates [m] : E490211 N6024306

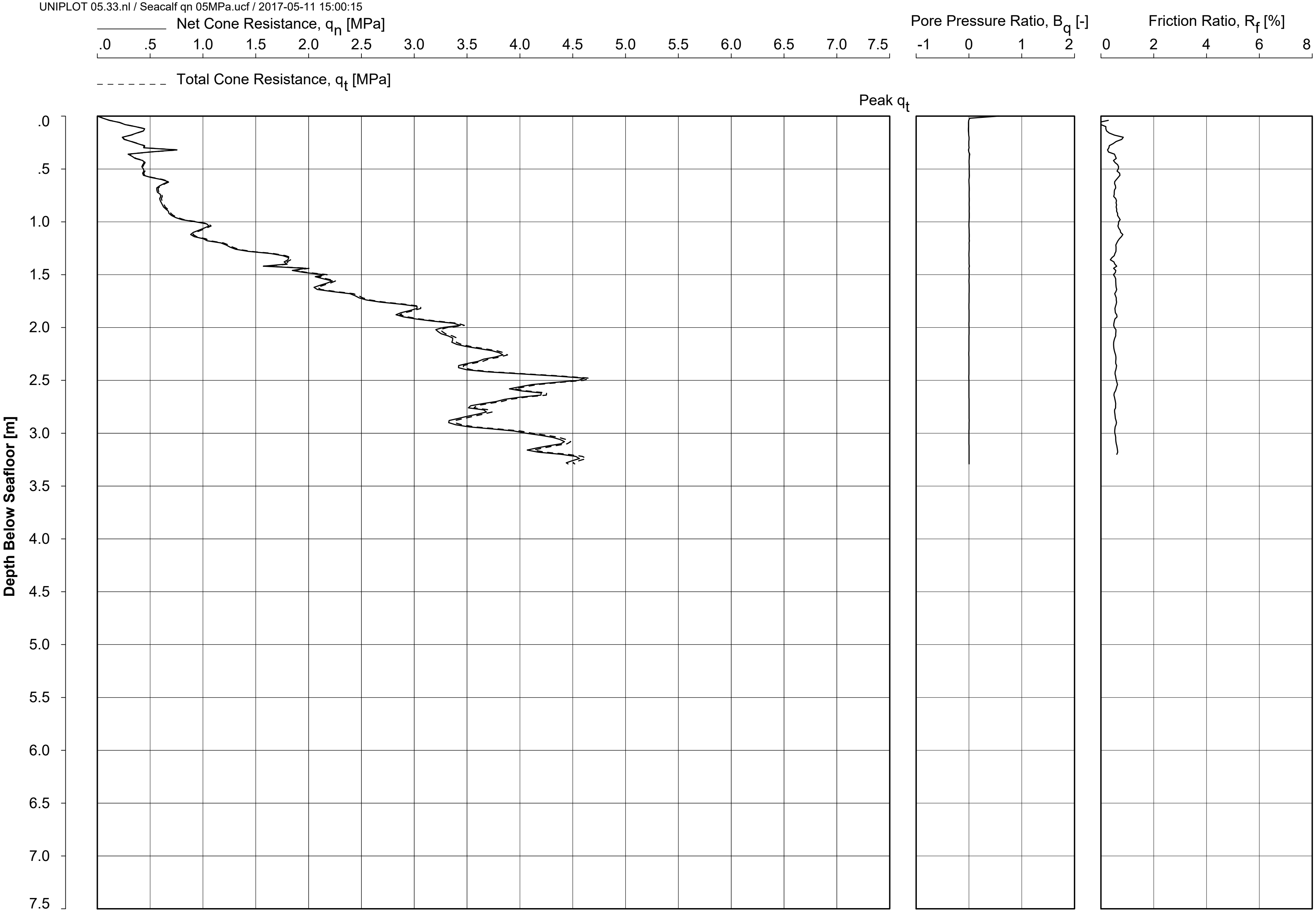
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT17

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



Date Of Testing : 20-Apr-2017
Water Depth [m] : 30.8
Coordinates [m] : E490619 N6023393

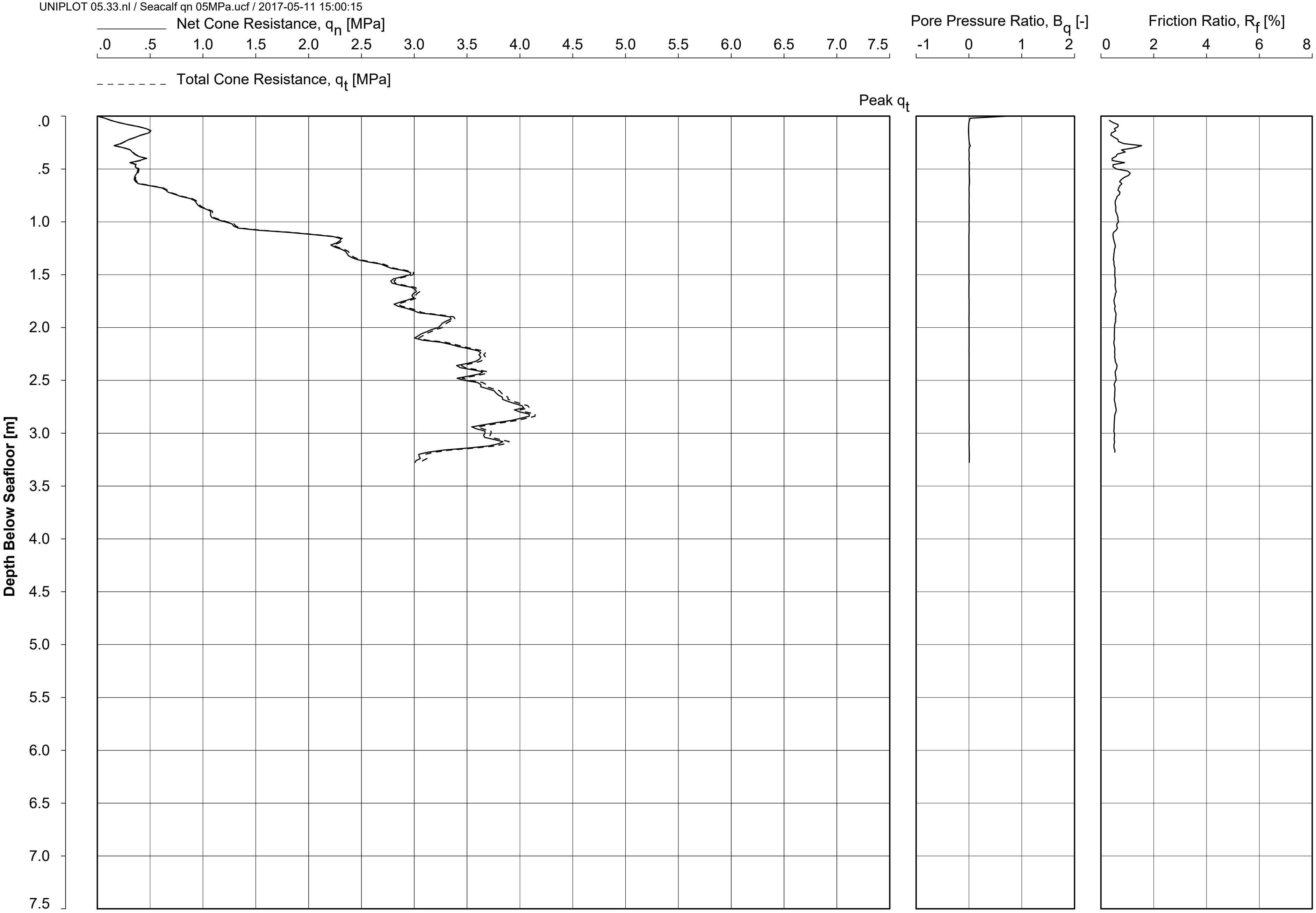
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT18

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf qn 05MPa.ucf / 2017-05-11 15:00:15



Date Of Testing : 20-Apr-2017
Water Depth [m] : 31.4
Coordinates [m] : E491029 N6022481

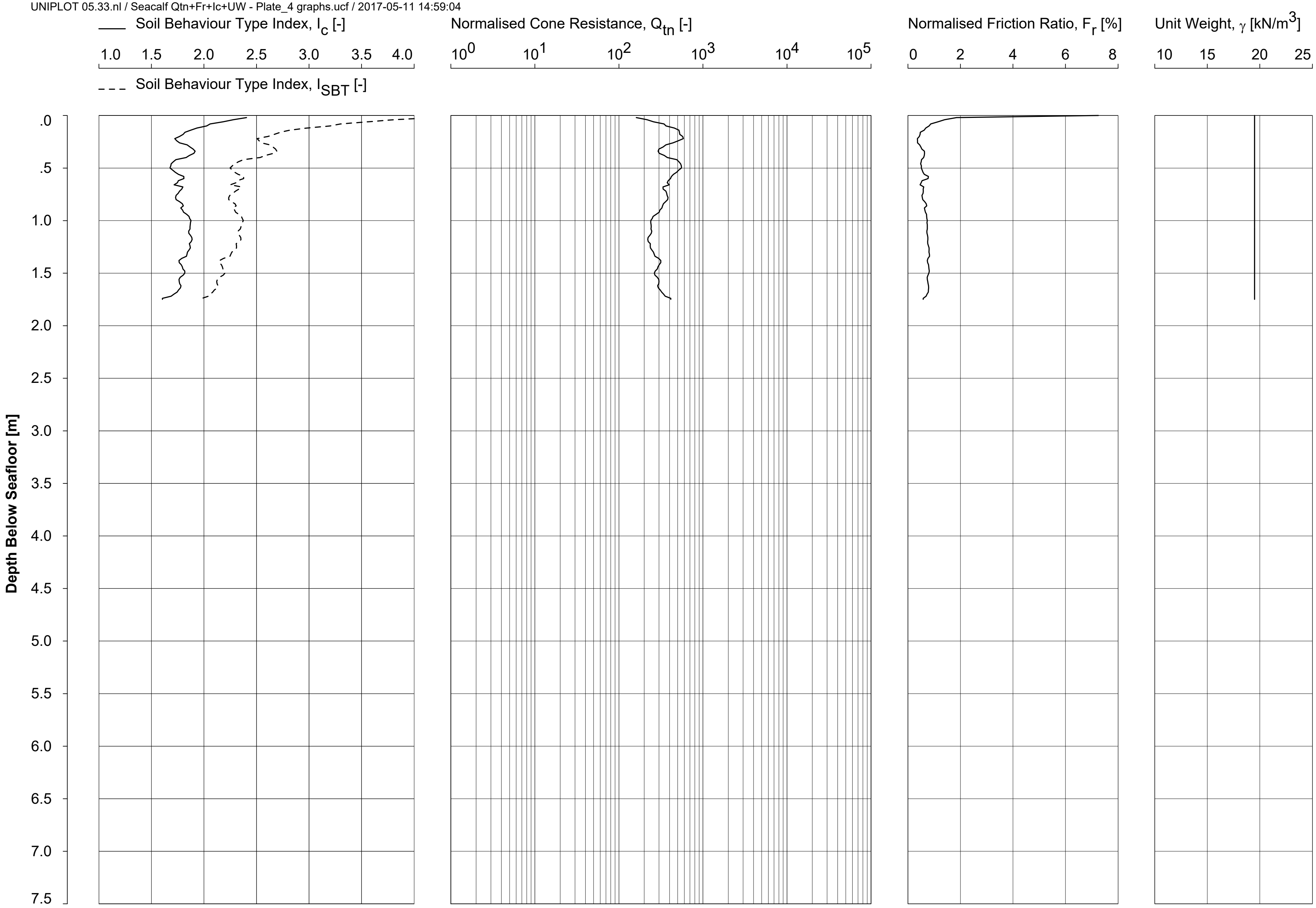
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503

CONE PENETRATION TEST

CPT19

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:04



Date of Testing : 19-Apr-2017
Water Depth [m] : 28.6
Coordinates [m] : E488162 N6028866

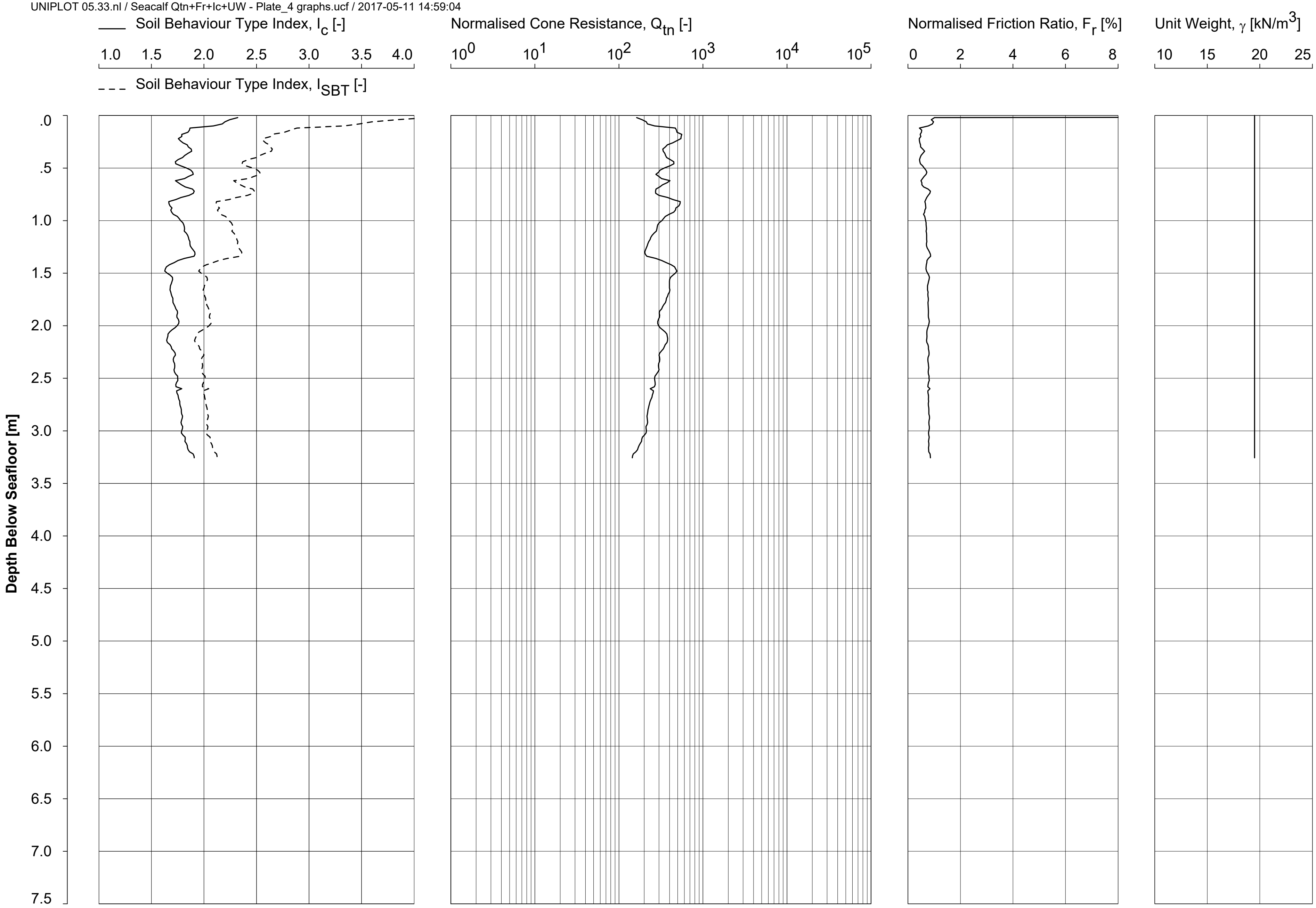
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT01

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:04



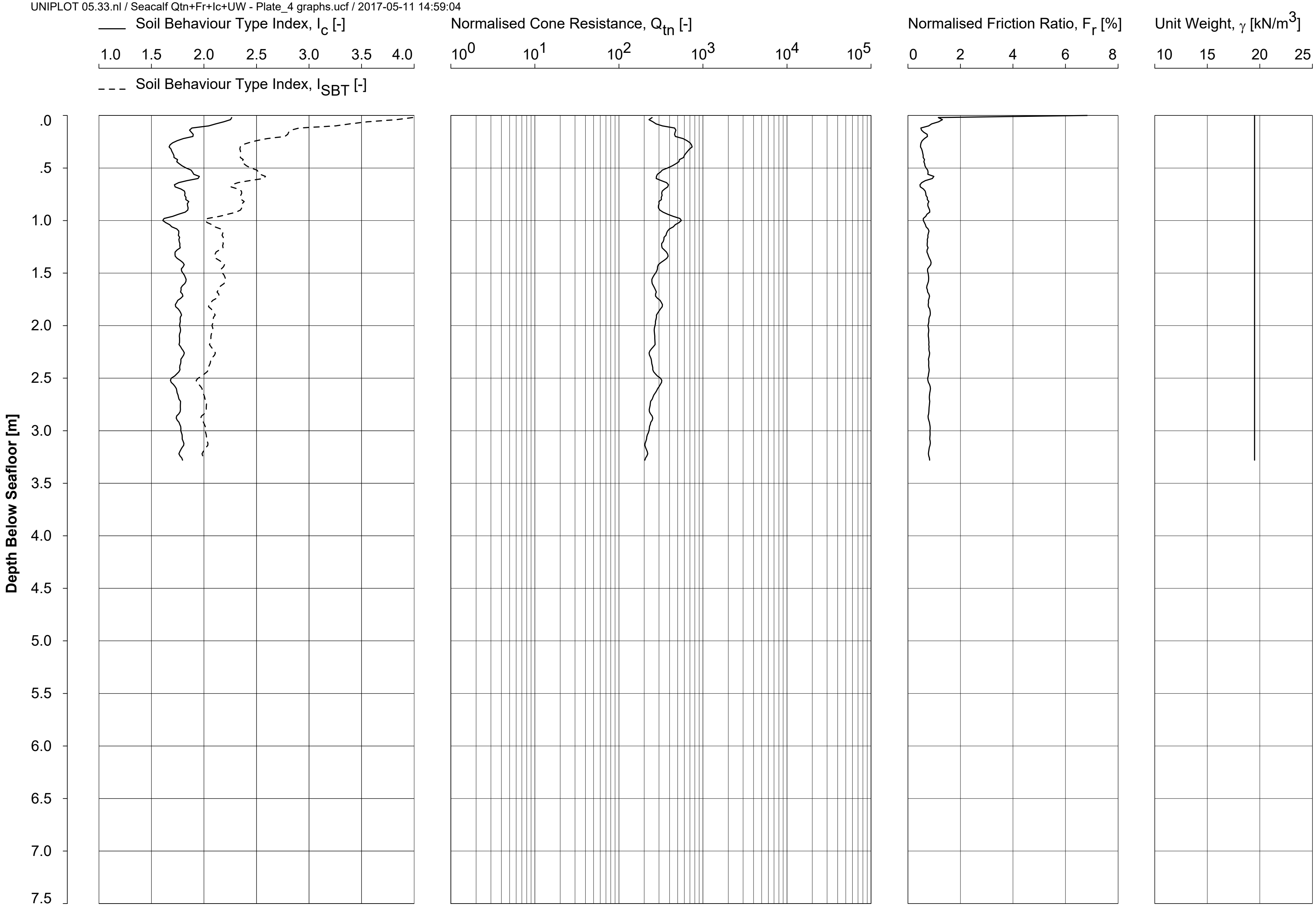
Date of Testing	: 19-Apr-2017	Probe Type	: CP5-CF35PB25SN2
Water Depth [m]	: 28.9	Cone Base Area [mm^2]	: 503
Coordinates [m]	: E488807 N6028105	Ground Water [kN/m^3]	: 10.0

CONE PENETRATION TEST

CPT02

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:04



Date of Testing : 19-Apr-2017
Water Depth [m] : 29.3
Coordinates [m] : E489454 N6027342

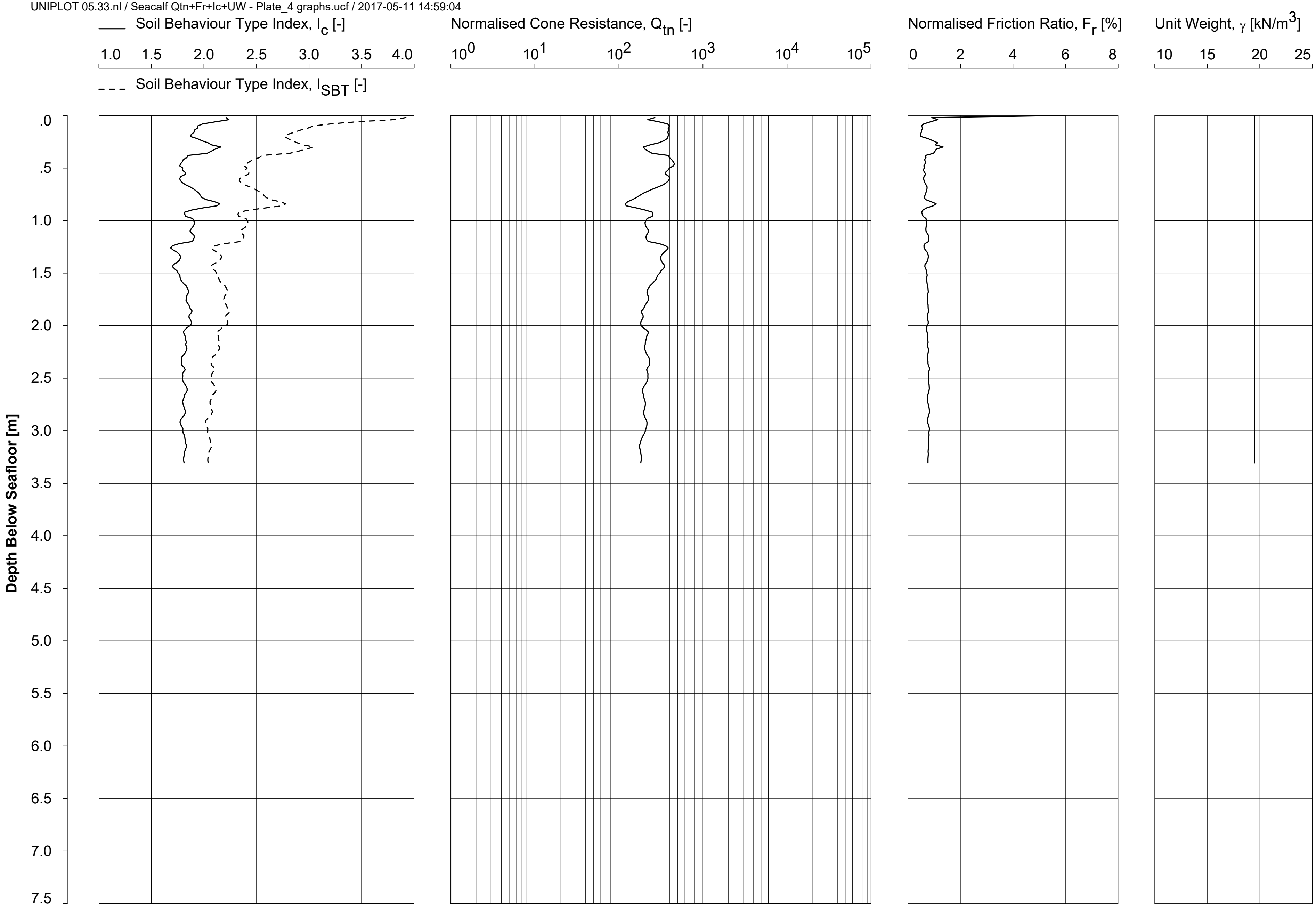
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT03

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:04



Date of Testing : 19-Apr-2017
Water Depth [m] : 29.6
Coordinates [m] : E490101 N6026576

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT04

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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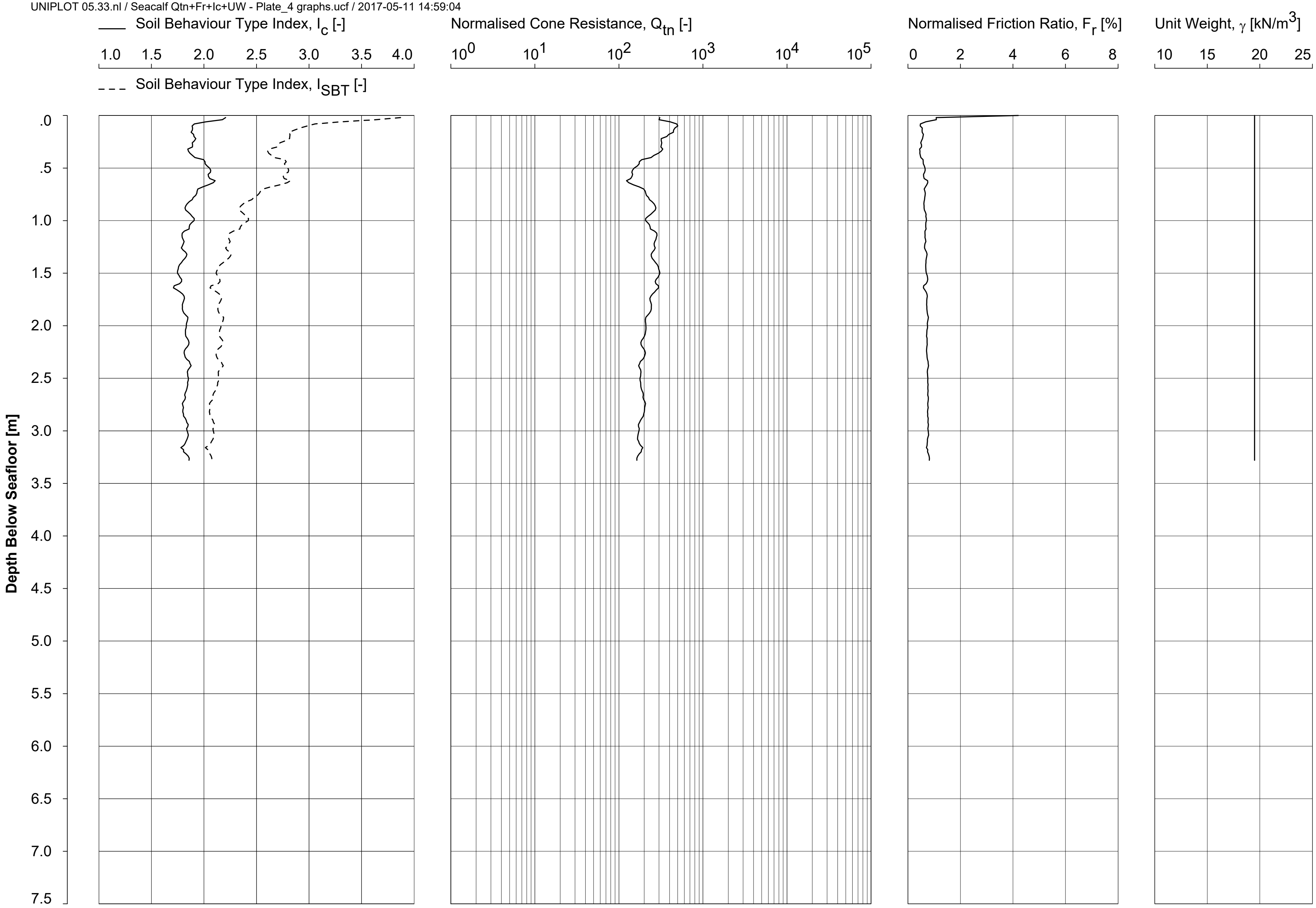

Date of Testing : 19-Apr-2017
Water Depth [m] : 30.0
Coordinates [m] : E490747 N6025815

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT05

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

FUGRO

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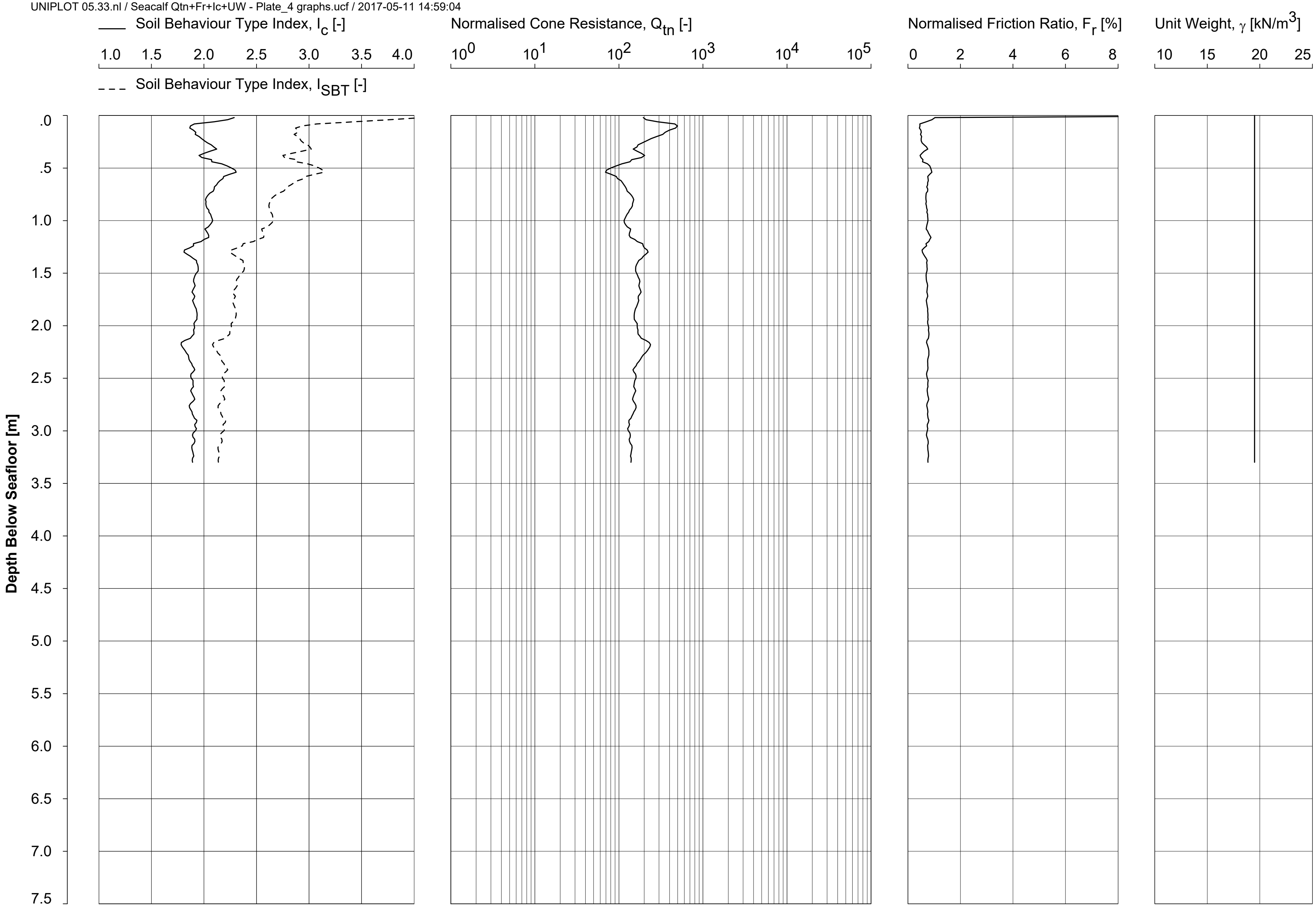

Date of Testing : 19-Apr-2017
Water Depth [m] : 30.6
Coordinates [m] : E491394 N6025052

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT06

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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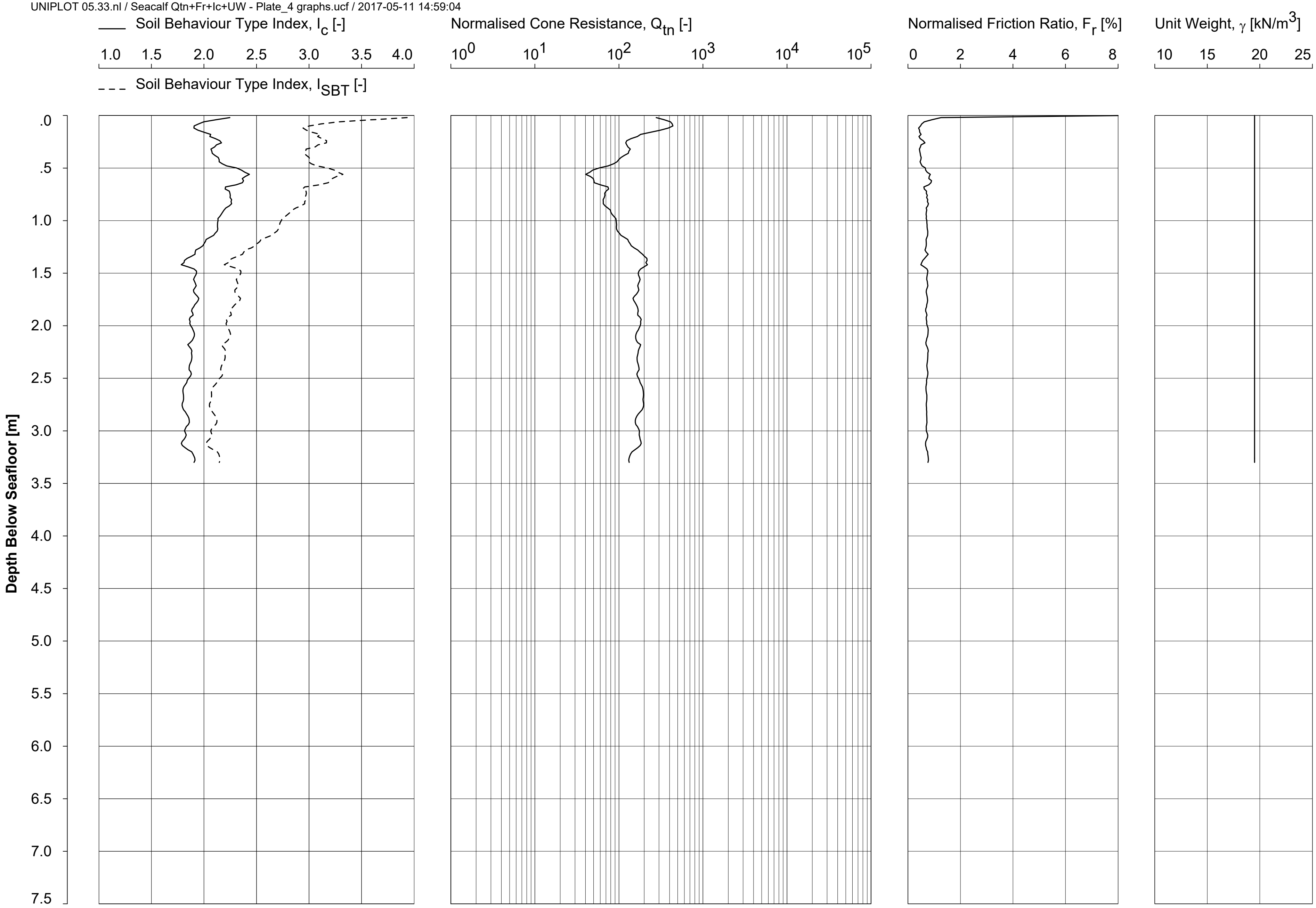

Date of Testing : 19-Apr-2017
Water Depth [m] : 31.2
Coordinates [m] : E492040 N6024287

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT07

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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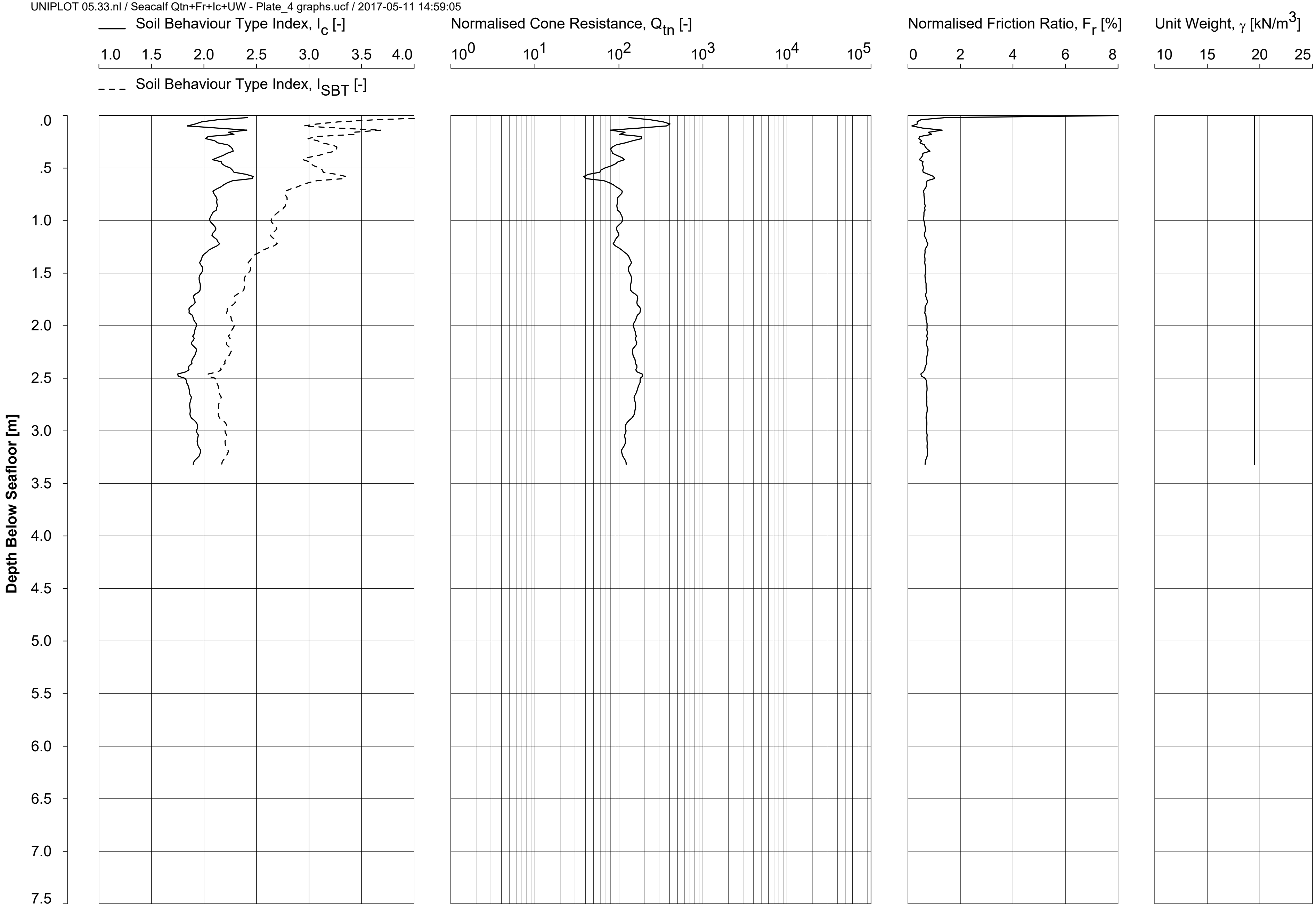

Date of Testing : 19-Apr-2017
Water Depth [m] : 31.9
Coordinates [m] : E492687 N6023526

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT08

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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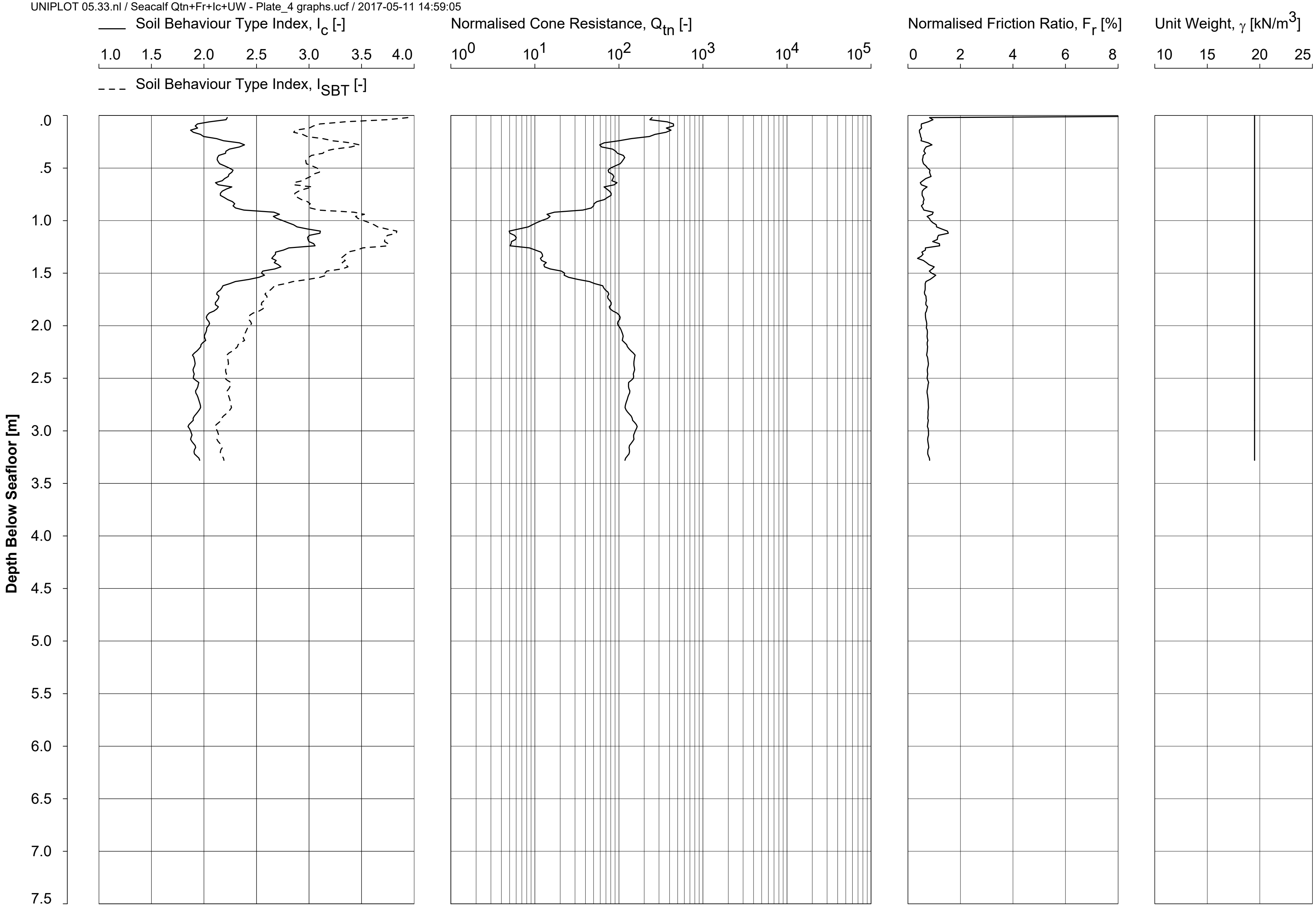

Date of Testing : 19-Apr-2017
Water Depth [m] : 33.1
Coordinates [m] : E493327 N6022770

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT09

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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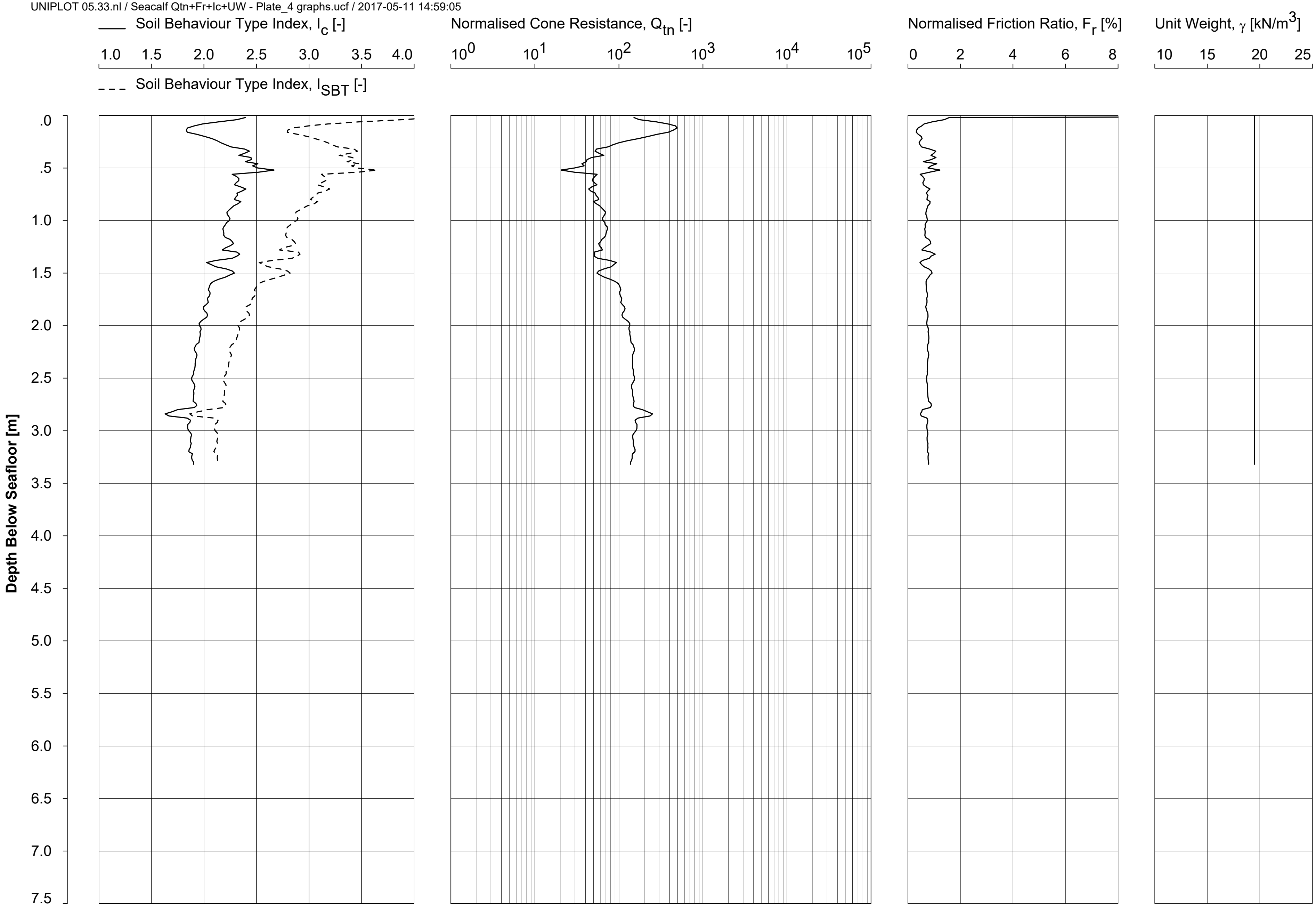

Date of Testing : 19-Apr-2017
Water Depth [m] : 34.6
Coordinates [m] : E493979 N6021999

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT10

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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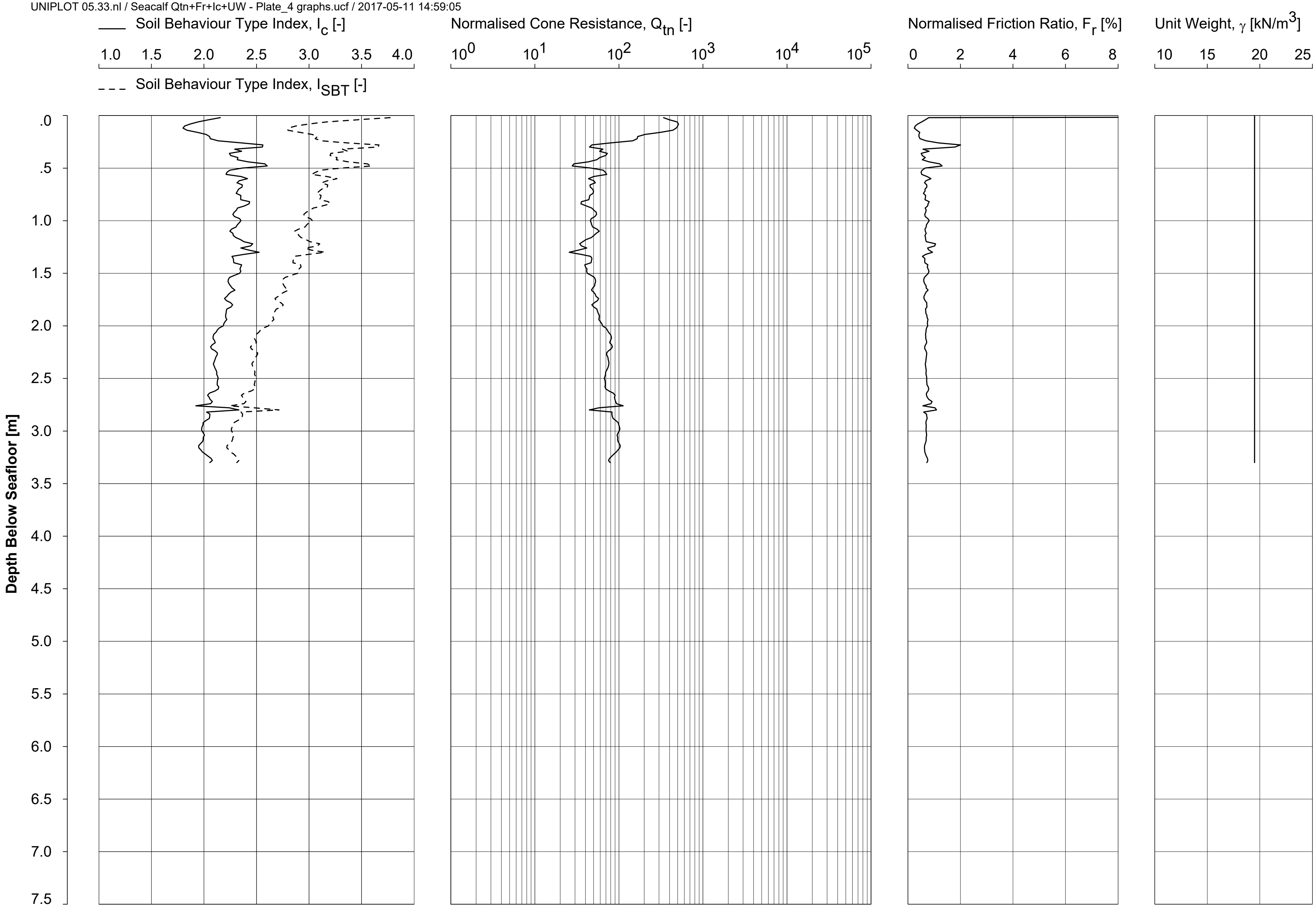

Date of Testing : 19-Apr-2017
Water Depth [m] : 36.3
Coordinates [m] : E494625 N6021237

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT11

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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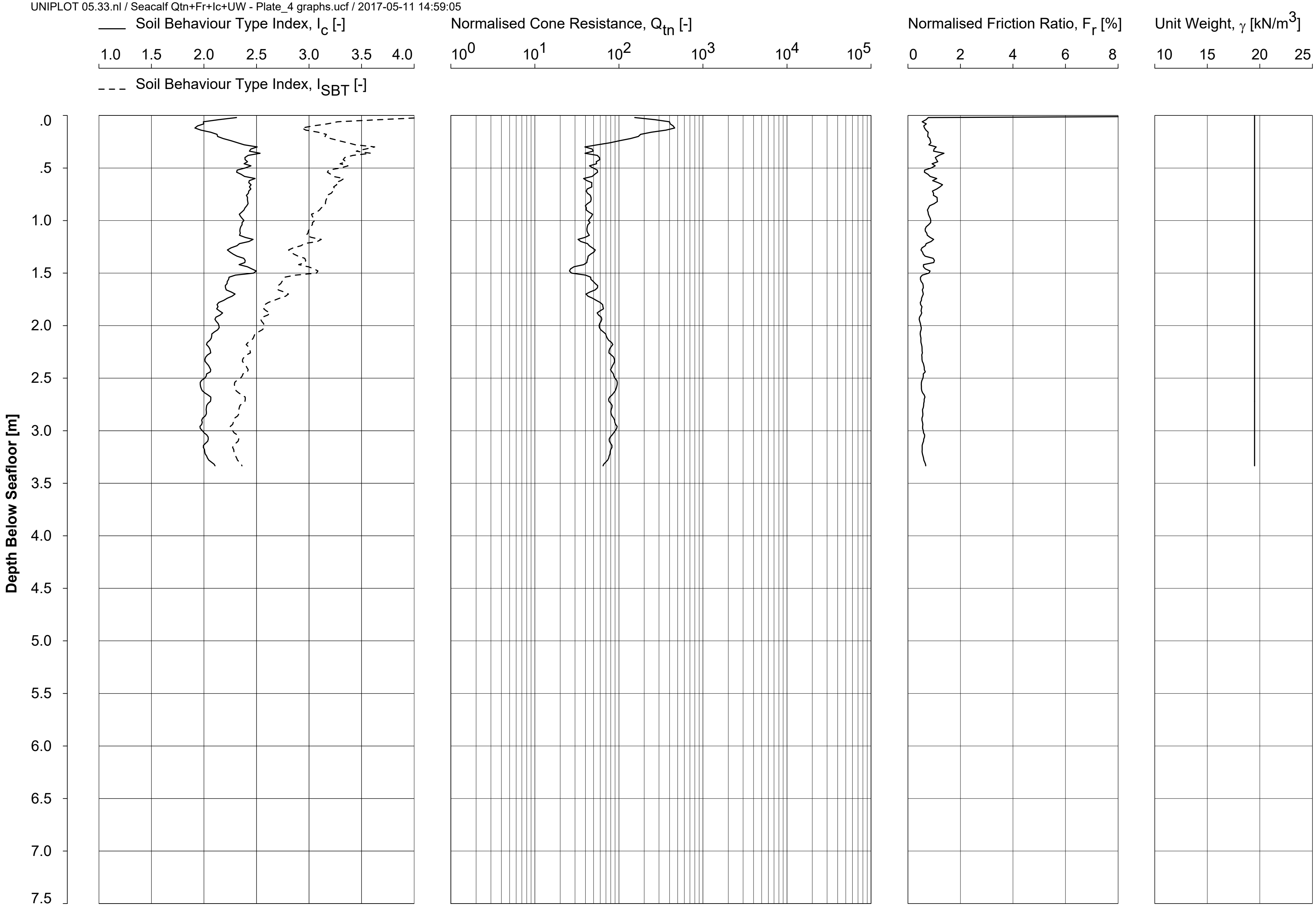

Date of Testing : 20-Apr-2017
Water Depth [m] : 38.2
Coordinates [m] : E495273 N6020475

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT12

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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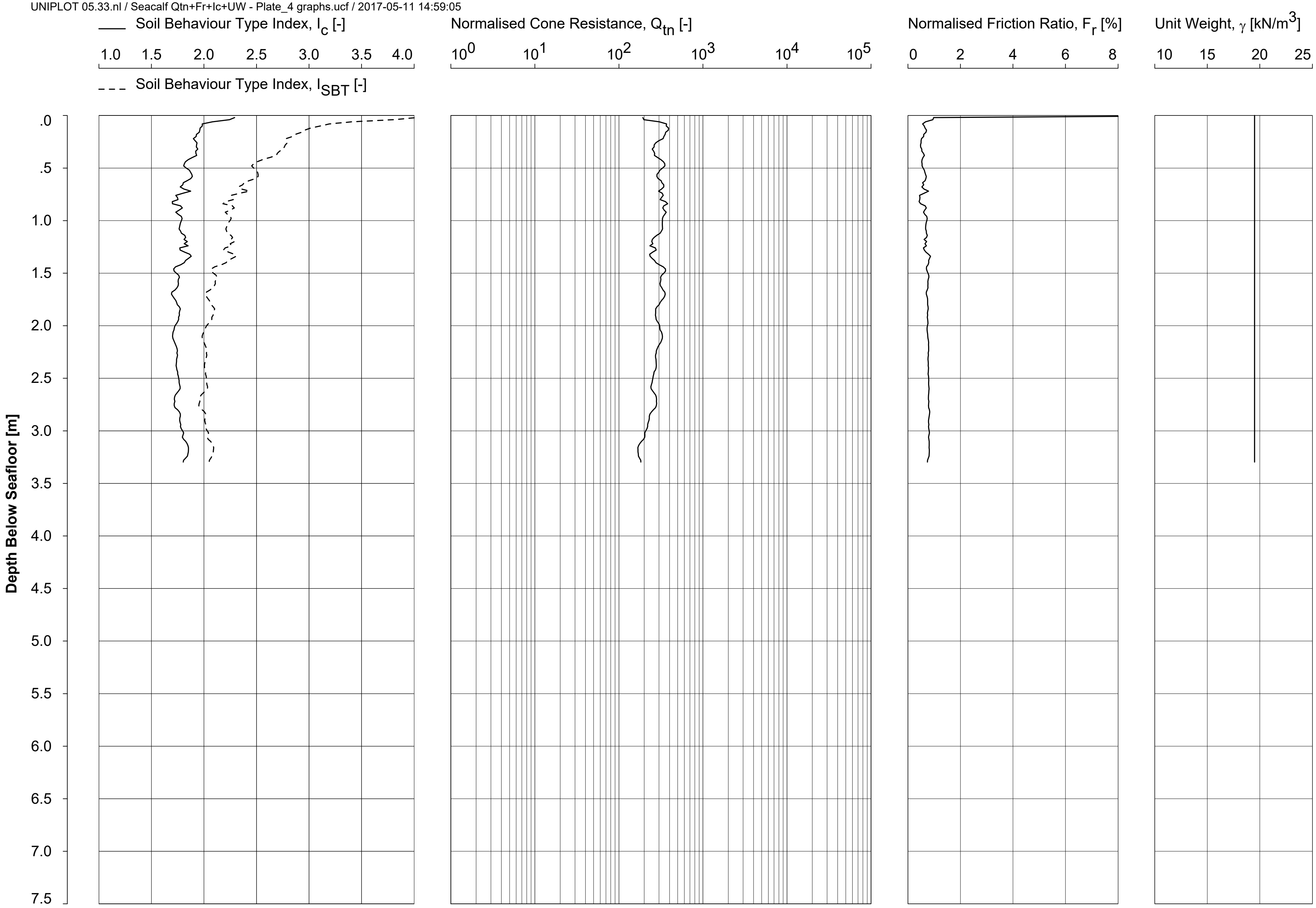


Date of Testing : 19-Apr-2017
Water Depth [m] : 28.8
Coordinates [m] : E488571 N6027954

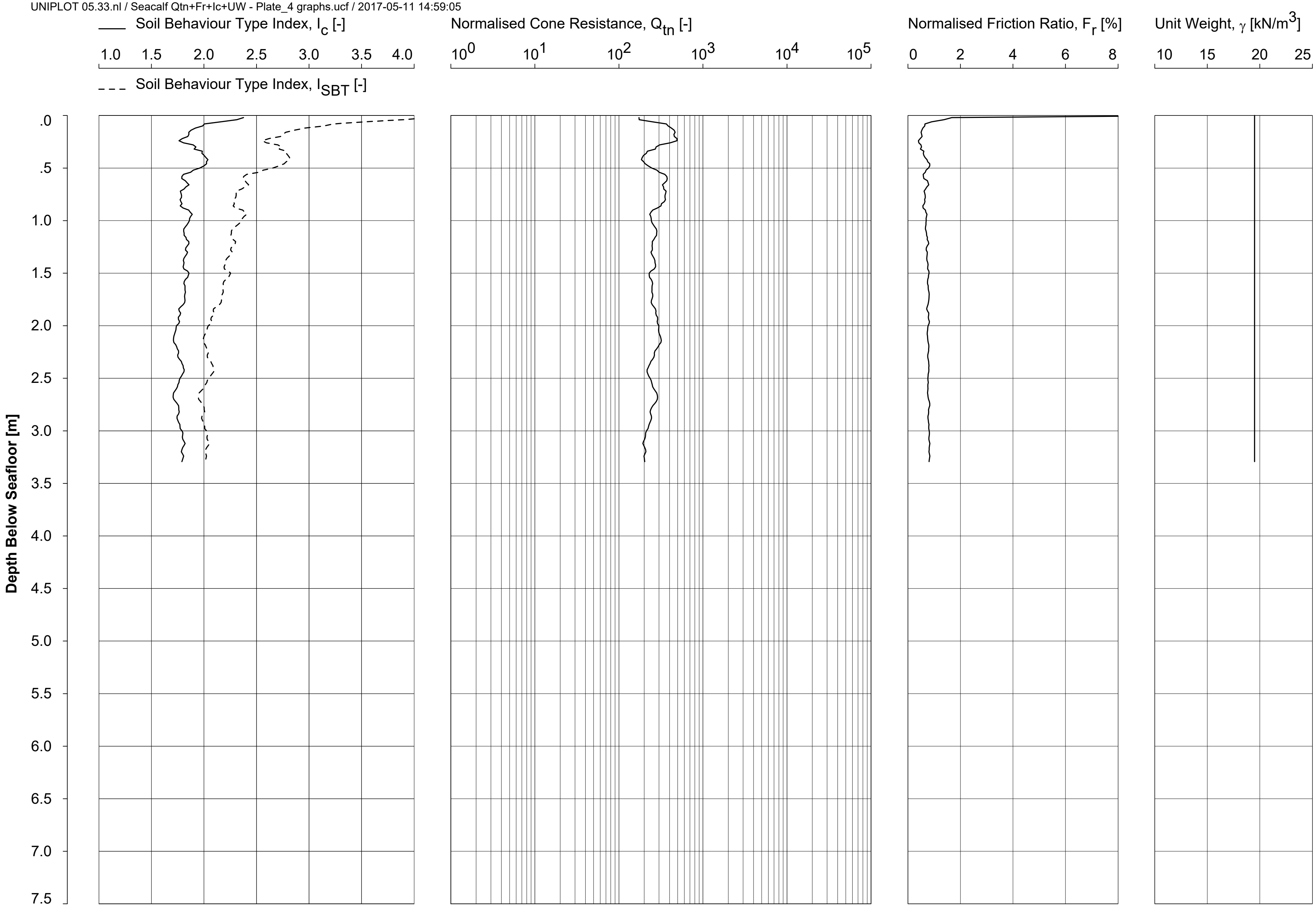
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT13

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:05



Date of Testing : 19-Apr-2017
Water Depth [m] : 29.1
Coordinates [m] : E488980 N6027041

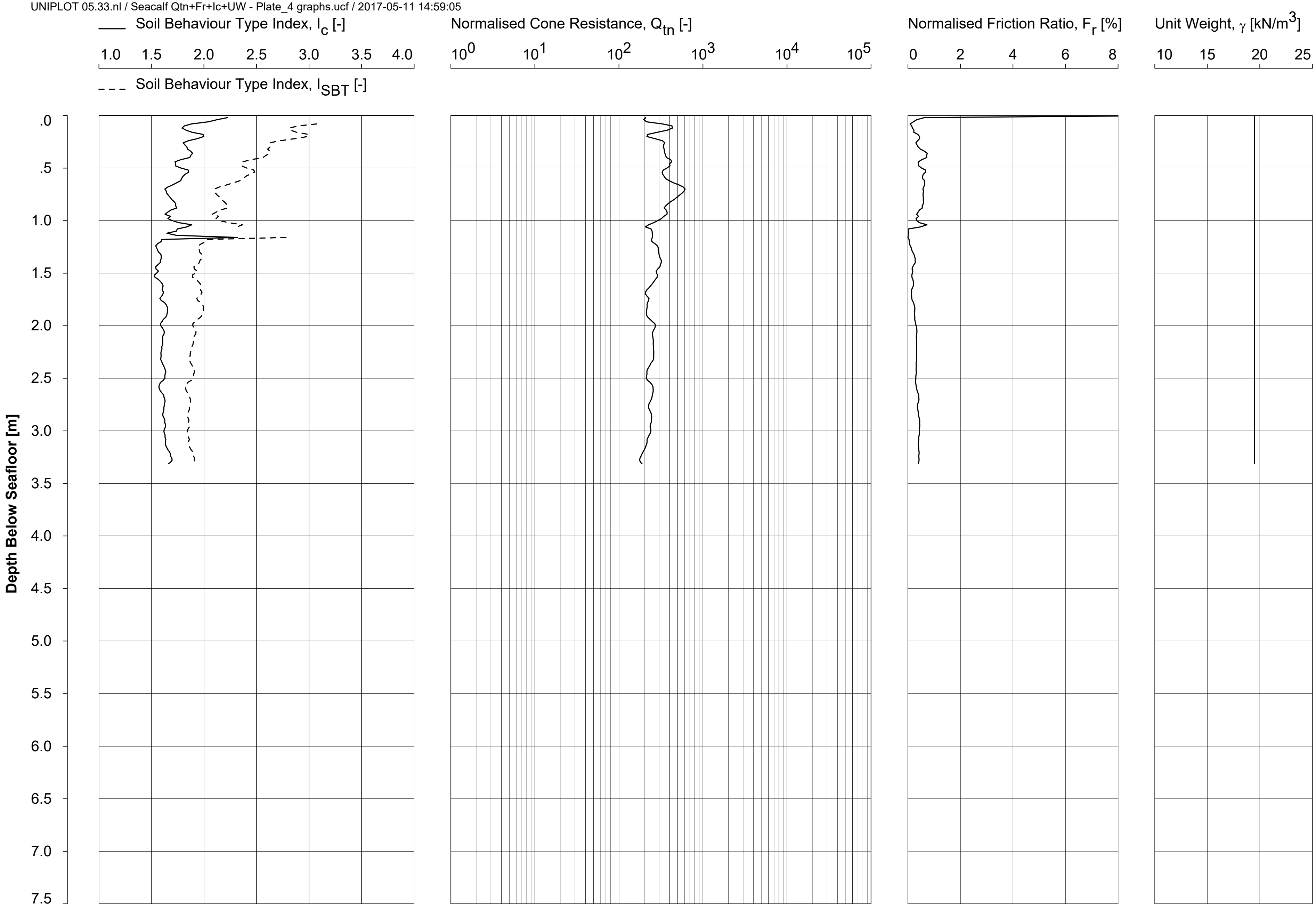
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT14

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:05



Date of Testing : 20-Apr-2017
Water Depth [m] : 29.4
Coordinates [m] : E489403 N6026101

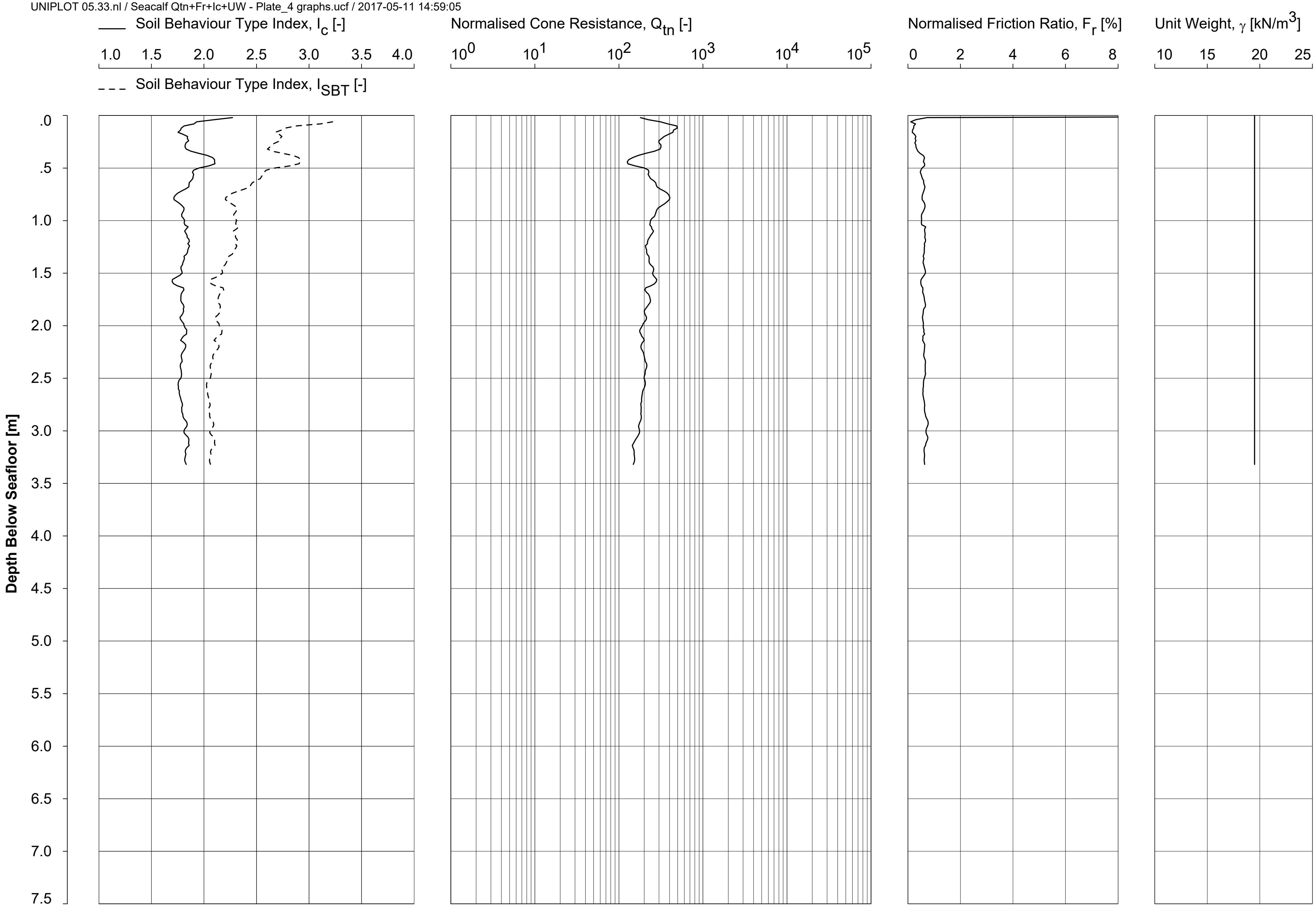
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT15

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:05



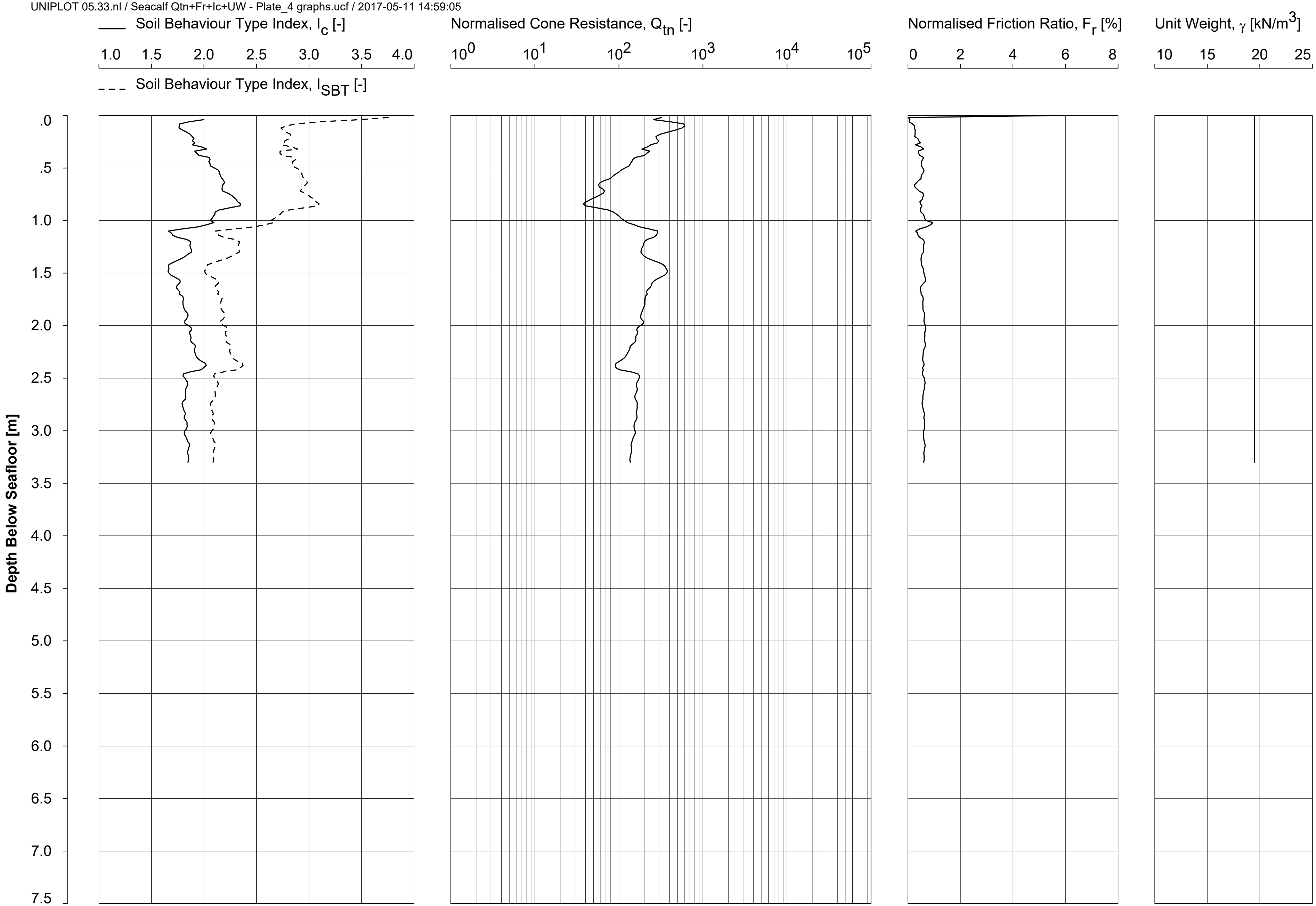
Date of Testing	: 20-Apr-2017	Probe Type	: CP5-CF35PB25SN2
Water Depth [m]	: 29.8	Cone Base Area [mm^2]	: 503
Coordinates [m]	: E489801 N6025217	Ground Water [kN/m^3]	: 10.0

CONE PENETRATION TEST

CPT16

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:05



Date of Testing : 20-Apr-2017
Water Depth [m] : 30.3
Coordinates [m] : E490211 N6024306

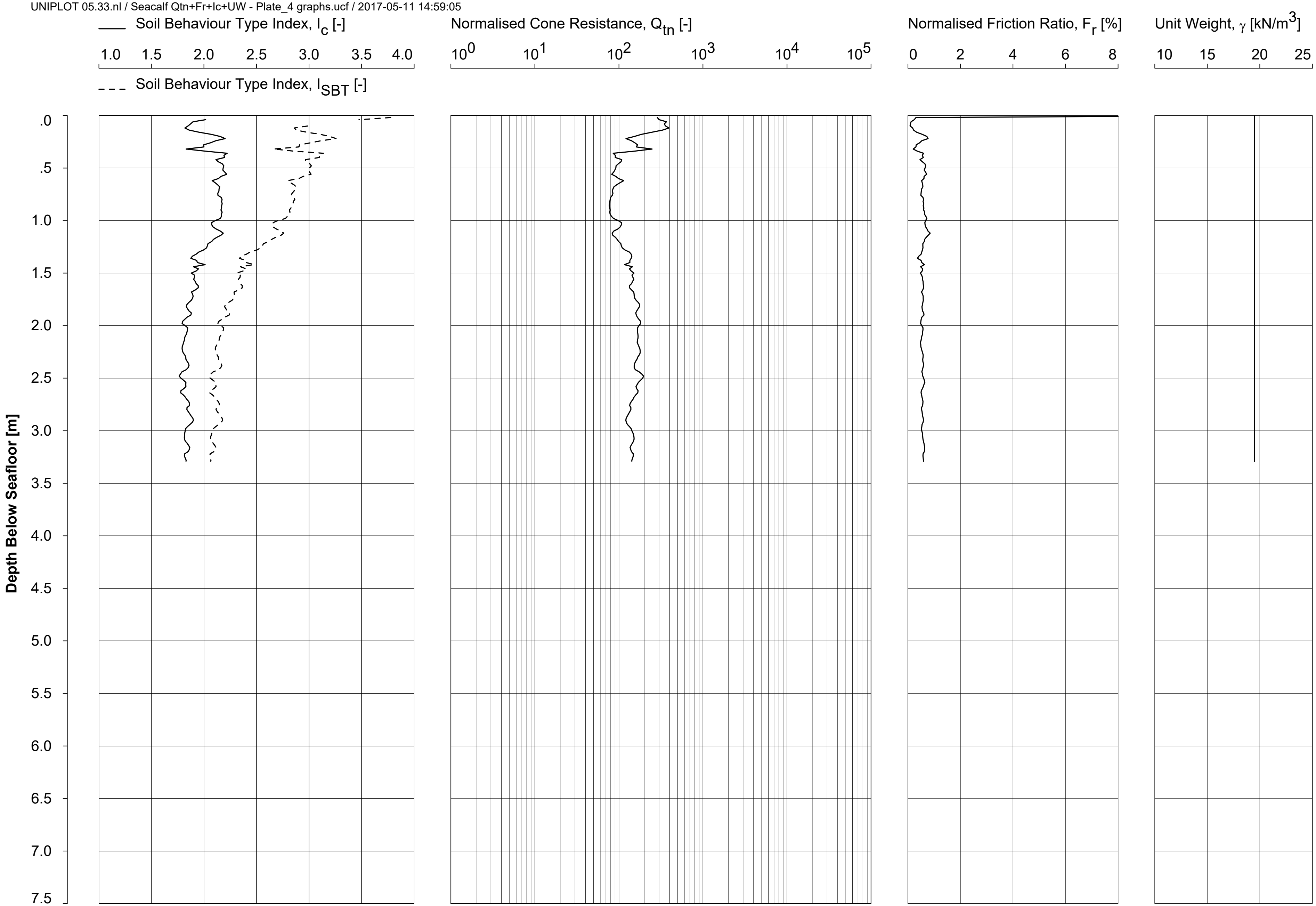
Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT17

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

UNIPLOT 05.33.nl / Seacalf Qtn+Fr+Ic+UW - Plate_4 graphs.ucf / 2017-05-11 14:59:05



Date of Testing : 20-Apr-2017
Water Depth [m] : 30.8
Coordinates [m] : E490619 N6023393

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT18

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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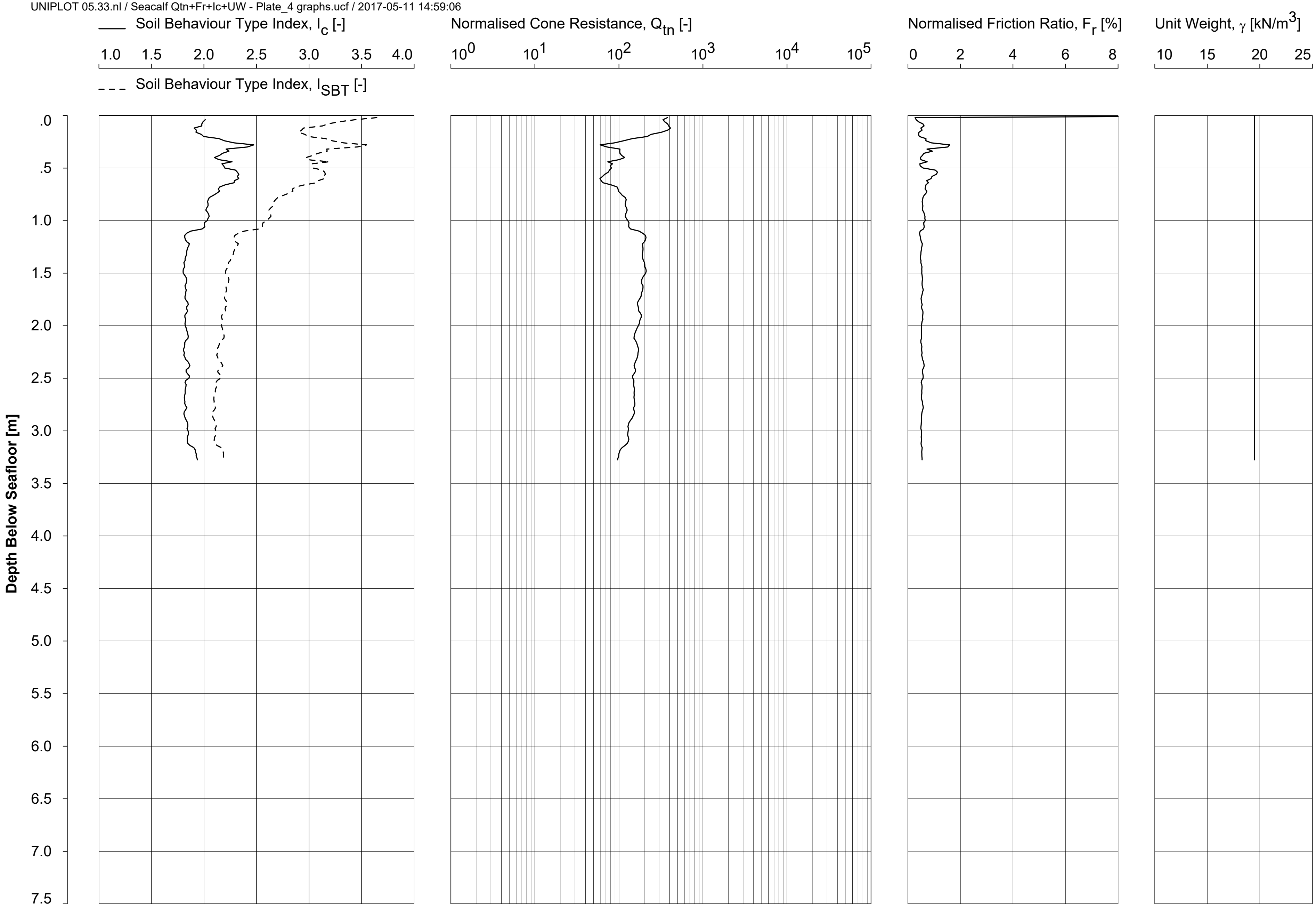

Date of Testing : 20-Apr-2017
Water Depth [m] : 31.4
Coordinates [m] : E491029 N6022481

Probe Type : CP5-CF35PB25SN2
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

CPT19

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

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CONE PENETRATION TEST - ZERO DRIFT

SILLIMANITE PIEPELINE ROUTES - DUTCH SECTOR, NORTH SEA

Borehole/ Location	Zero Reading at Start of Test			Zero Drift			Probe		Net Area Ratio
	q_c [MPa]	f_s [MPa]	u [MPa]	q_c [MPa]	f_s [MPa]	u [MPa]			a [-]
CPT01	0.421	-0.008	-0.123	-0.022	0.010	-0.008	CP5-CF35PB25SN2	1717-1494	0.500
CPT02	0.404	0.001	-0.128	-0.011	-0.002	-0.004	CP5-CF35PB25SN2	1717-1494	0.500
CPT03	0.397	-0.014	-0.132	-0.017	0.000	0.003	CP5-CF35PB25SN2	1717-1494	0.500
CPT04	0.388	-0.014	-0.137	-0.011	0.000	0.000	CP5-CF35PB25SN2	1717-1494	0.500
CPT05	0.393	-0.014	-0.130	-0.010	0.000	-0.003	CP5-CF35PB25SN2	1717-1494	0.500
CPT06	0.392	-0.014	-0.129	-0.011	0.000	-0.007	CP5-CF35PB25SN2	1717-1494	0.500
CPT07	0.395	-0.014	-0.129	-0.007	0.000	-0.009	CP5-CF35PB25SN2	1717-1494	0.500
CPT08	0.396	-0.014	-0.118	-0.011	0.000	-0.016	CP5-CF35PB25SN2	1717-1494	0.500
CPT09	0.395	-0.014	-0.125	-0.011	0.009	-0.008	CP5-CF35PB25SN2	1717-1494	0.500
CPT10	0.397	-0.014	-0.122	-0.009	0.000	-0.006	CP5-CF35PB25SN2	1717-1494	0.500
CPT11	0.391	-0.014	-0.132	0.003	0.000	-0.001	CP5-CF35PB25SN2	1717-1494	0.500
CPT12	0.182	0.005	0.015	0.004	0.000	0.000	CP5-CF35PB25SN2	1717-1645	0.500
CPT13	0.400	-0.015	-0.124	-0.012	0.000	-0.006	CP5-CF35PB25SN2	1717-1494	0.500
CPT14	0.396	-0.015	-0.132	-0.009	0.000	0.001	CP5-CF35PB25SN2	1717-1494	0.500
CPT15	0.003	0.020	0.020	0.156	-0.014	0.002	CP5-CF35PB25SN2	1717-1645	0.500
CPT16	0.168	0.006	0.019	-0.009	0.000	0.001	CP5-CF35PB25SN2	1717-1645	0.500
CPT17	0.169	0.006	0.017	-0.009	0.000	0.000	CP5-CF35PB25SN2	1717-1645	0.500
CPT18	0.171	0.005	0.015	-0.007	0.000	0.002	CP5-CF35PB25SN2	1717-1645	0.500
CPT19	0.171	0.005	0.016	-0.007	0.001	0.001	CP5-CF35PB25SN2	1717-1645	0.500

Key:
q_c : cone resistance f_s : sleeve friction u : pore water pressure

Note:
1. Zero Drift is the difference between the zero output at the start of the test and the zero output at the end of the test. Offshore tests may show Reference Readings. The Zero Reading or Reference Reading at Start of Test is a value presented in units of measurement result. The value itself is a conversion from system output, usually in mV. It has no explicit physical meaning.
2. --- : Zero Drift was not monitored. The drift can be assessed from the start values of successive tests.

SECTION C: USE OF REPORT

CONTENTS Reference

DOCUMENT ISSUE CONTROL

Section	Page No.	Plate No.	Issue	Revision

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QUALITY MANAGEMENT RECORD

Fugro Project Lead:	For J. Lakeman Senior Project Engineer	Signed:	A.A. Duijster
Report Review and Approval:	M. Klein Principal Geotechnical Engineer	Signed:	

Document Section		Prepared By	Checked By
Main Text		CBO/DBW/TAD	JLK
Plates following Main Text		CBO/DBW/TAD	JLK
A	Geotechnical Logs		
	– Borehole Logs	CBO/DBW/TAD	JLK
B	In Situ Test Results		
	– Cone Penetration Tests	CBO/DBW/TAD	JLK

Person(s):

CBO C. Fremouw-Bost
DBW D. Brouwer
JLK J. Lakeman
TAD A. Duijster

INTRODUCTION

This document provides guidelines, recommendations and limitations regarding the use of information in this report.

The cost of geotechnical data acquisition, interpretation and monitoring is a small portion of the total cost of a construction project. By contrast, the costs of correcting a wrongly designed programme or mobilising alternative construction methods are often far greater than the cost of the original investigation. Attention and adherence to the guidelines and recommendations presented in this guide and in the geotechnical report can reduce delays and cost overruns related to geotechnical factors.

This guide applies equally to the use of geotechnical and multi-disciplinary project information and advice.

REQUIREMENTS FOR QUALITY GEOTECHNICAL INFORMATION

Fugro follows ISO 9001 quality principles for project management and ISO 2394 for general principles on reliability for structures. Project activities usually comprise part of specific phases of a construction project. The quality plan for the entire construction project should incorporate geotechnical input in every phase - from the feasibility planning stages to project completion. The parties involved should do the following:

- Provide complete and accurate information necessary to plan an appropriate geotechnical site investigation.
- Describe the purpose(s), type(s) and construction methods of planned structures in detail.
- Provide the time, financial, personnel and other resources necessary for the planning, execution and follow-up of a site investigation programme.
- Understand the limitations and degree of accuracy inherent in the geotechnical data and engineering advice based upon these data.
- During all design and construction activities, be aware of the limitations of geotechnical data and geotechnical engineering analyses/advice, and use appropriate preventative measures.
- Incorporate all geotechnical input in the design, planning, construction and other activities involving the site and structures. Provide the entire geotechnical report to parties involved in design and construction.
- Use the geotechnical data and engineering advice for only the structures, site and activities which were described to Fugro prior to and for the purpose of planning the geotechnical site investigation or geotechnical engineering analysis programme.

AUTHORITY, TIME AND RESOURCES NECESSARY FOR GEOTECHNICAL INVESTIGATIONS

Adequate designation of authority and accountability for geotechnical aspects of construction projects is necessary. This way, an appropriate investigation can be performed, and the use of the results by project design and construction professionals can be optimised.

Figure 1 illustrates the importance of the initial project phases for gathering adequate geotechnical information for a project. The initial phases, when site investigation requirements are defined and resources are allocated, are represented by more than 50% of the Quality Triangle (Figure 1). Decisions and actions made during these phases have a large impact of the outcome and thus the potential of the investigation to meet project requirements.

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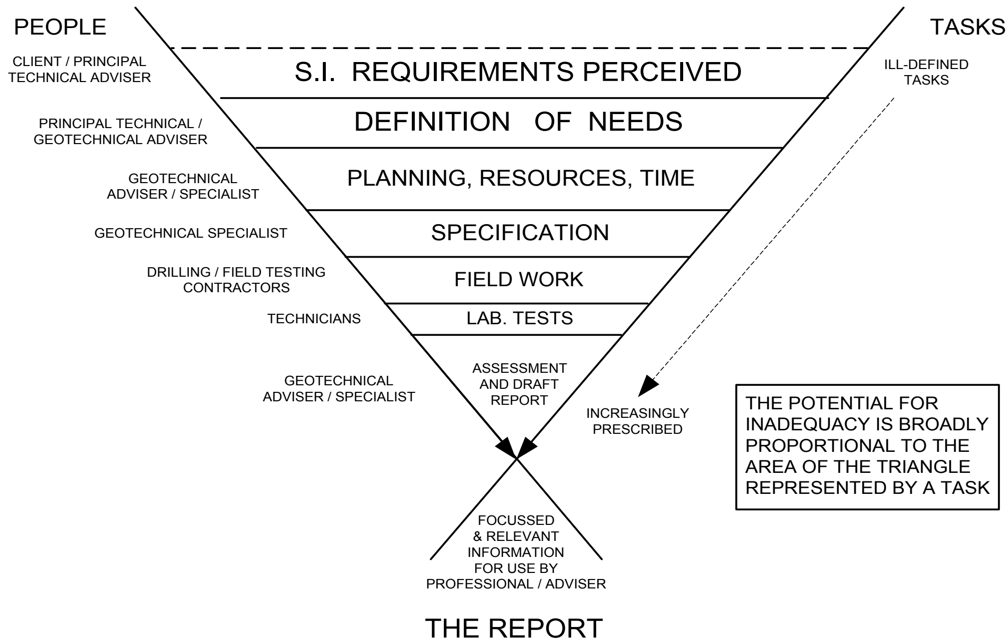
Figure 1: Quality of Geotechnical Site Investigation (adapted from SISG[1]).



DATA ACQUISITION AND MONITORING PROGRAMMES

Geotechnical investigations are operations of discovery. Investigation should proceed in logical stages. Planning should allow operational adjustments deemed necessary by newly available information. This observational approach permits the development of a sound engineering strategy and reduces the risk of discovering unexpected hazards during or after construction.

GEOTECHNICAL INFORMATION – DATA TYPES AND LIMITATIONS

1. RELIABILITY OF SUPPLIED INFORMATION

Geotechnical engineering can involve the use of information and physical material that is publicly available or supplied by the Client. Examples are geodetic data, geological maps, geophysical records, earthquake data, earlier geotechnical logs and soil samples. Fugro endeavours to identify potential anomalies, but does not independently verify the accuracy or completeness of public or Client-supplied information unless indicated otherwise. This information, therefore, can limit the accuracy of the report.

2. COMPLEXITY OF GROUND CONDITIONS

There are hazards associated with the ground. An adequate understanding of these hazards can help to minimize risks to a project and the site. The ground is a vital element of all structures which rest on or in the ground. Information about ground behaviour is necessary to achieve a safe and economical structure. Often less is known about the ground than for any other element of a structure.

3. GEOTECHNICAL INVESTIGATION - SPATIAL COVERAGE LIMITATIONS

Geotechnical investigations collect data at specific test locations. Interpretation of ground conditions away from test locations is a matter of extrapolation and judgement based on geotechnical knowledge and experience, but actual conditions in untested areas may differ from predictions. For example, the interface between ground materials may be far more gradual or abrupt than a report indicates. It is not realistic to expect a geotechnical investigation to reveal or anticipate every detail of ground conditions. Nevertheless, an investigation can reduce the residual risk associated with unforeseen conditions to a tolerable level. If ground problems do arise, it is important to have geotechnical expertise available to help reduce and mitigate safety and financial risks.

1 Site Investigation Steering Group SISG 1993. *Site Investigation in Construction 2: Planning, Procurement and Quality Management*. London: Thomas Telford.

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4. ROLE OF JUDGEMENT AND OPINION IN GEOTECHNICAL ENGINEERING

Geotechnical engineering is less exact than most other design disciplines, and requires extensive judgement and opinion. Therefore, a geotechnical report may contain definitive statements that identify where the responsibility of Fugro begins and ends. These are not exculpatory clauses designed to transfer liabilities to another party, but they are statements that can help all parties involved to recognise their individual responsibilities and take appropriate actions.

COMPLETE GEOTECHNICAL REPORT SHOULD BE AVAILABILE TO ALL PARTIES INVOLVED

To prevent costly construction problems, construction contractors should have access to the best available information. They should have access to the complete original report to prevent or minimize any misinterpretation of site conditions and engineering advice. To prevent errors or omissions that could lead to misinterpretation, geotechnical logs and illustrations should not be redrawn, and users of geotechnical engineering information and advice should confer with the authors when applying the report information and/or recommendations.

GEOTECHNICAL INFORMATION IS PROJECT-SPECIFIC

Fugro's investigative programmes and engineering assessments are designed and conducted specifically for the Client described project and conditions. Thus this report presents data and/or recommendations for a unique construction project. Project-specific factors for a structure include but are not limited to:

- location
- size and configuration of structure
- type and purpose or use of structure
- other facilities or structures in the area.

Any factor that changes subsequent to the preparation of this report may affect its applicability. A specialised review of the impact of changes would be necessary. Fugro is not responsible for conditions which develop after any factor in site investigation programming or report development changes.

For purposes or parties other than the original project or Client, the report may not be adequate and should not be used.

CHANGES IN SUBSURFACE CONDITIONS AFFECT THE ACCURACY / SUITABILITY OF THE DATA

Ground is complex and can be changed by natural phenomena such as earthquakes, floods, seabed scour and groundwater fluctuations. Construction operations at or near the site can also change ground conditions. This report considers conditions at the time of investigation. Construction decisions should consider any changes in site conditions, regulatory provisions, technology or economic conditions subsequent to the investigation. In general, two years after the report date, the information may be considered inaccurate or unreliable. A specialist should be consulted regarding the adequacy of this geotechnical report for use after any passage of time.

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APPENDIX 1: DESCRIPTIONS OF METHODS AND PRACTICES

CONTENTS Reference

This appendix presents method statements and terminology that are generally familiar to expert users of the information.

INTRODUCTION

A geotechnical log presents a one-dimensional, typically vertical, profile of ground strata and water level measurements, where applicable. In addition, it may include the principal details of operational activities for acquisition of the information shown on the geotechnical log. Other terms for geotechnical log used in practice include core log, borehole log, sample log, geohazard core log, geological log.

The format and detail of a geotechnical log depends on an agreed project specification. This means that the geotechnical log considers a specific purpose, for example presentation of geotechnical information for design of a pile foundation. The geotechnical log should not be used for another purpose without appropriate verification. A minimal geotechnical log can consist of descriptions limited to e.g. “soil” and “rock” or a value such as soil behaviour type index, in combination with corresponding depths below ground surface or seafloor. A comprehensive geotechnical log can include mm-scale geological descriptions, geotechnical strata, data points of laboratory test results and multiple profiles of geophysical borehole logging and interpretive results of in situ testing.

The level of detail and accuracy of a geotechnical log also depend on factors such as sample size, quality, coverage of samples and test data and availability of supplementary information.

Preparation of geotechnical logs is based on ISO, CEN, BSI and ASTM standards.

EXAMPLE INFORMATION - GEOTECHNICAL LOGS

OPERATIONAL ACTIVITIES

Additional documentation of operational activities can include further details on drilling, sampling and in situ testing. In particular, details of sampling techniques and samplers can be important for the evaluation of the results of laboratory tests. Figure 1 shows examples for presentation of operational information.

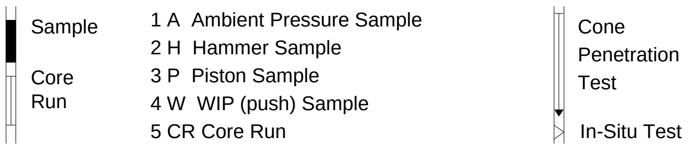

Figure 1 - Symbols for identification of samples and in situ tests

The penetration depth shown on a (vertical) geotechnical log is defined as the deepest point reached by drilling, sampling or in situ testing. The recovery depth is the deepest point for which logging, sample or test data are presented.

Unless indicated otherwise, recovery of a borehole tube sample or a core sample is assumed and shown to be continuous from the starting depth of sampling. Similarly, sample recovery for a seafloor sampler is assumed to be continuous from seafloor to recovery depth. In other words, the geotechnical log ignores possible plugging, flow-in and/or wash-out.

MWD PARAMETERS

Measurement while drilling (MWD) parameters for rotary drilling can help characterisation of ground conditions such as cemented strata, weak rock and formations with cavities apply. Presentation of factual and/or interpreted results is usually in graphical format.

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ROCK CORING PARAMETERS

ASTM International (2017) provides descriptions for rock core quality as follows:

TCR Total Core Recovery: the total core length divided by the core run length

SCR Solid Core Recovery: the total length of the pieces of solid core that have a complete circumference divided by the core run length

RQD Rock Quality Designation: the total length of the pieces of sound core over 100 mm long along the centreline divided by the core run lengths per stratum or core run; sound core includes core with obvious drilling breaks

I_F Fracture Index: spacing of natural discontinuities.

Table 1 shows a classification of rock quality according to ASTM International (2017).

TABLE 1 CLASSIFICATION OF ROCK QUALITY

RQD	Classification of Rock Quality
0 % to 25 %	Very poor
25 % to 50 %	Poor
50 % to 75 %	Fair
75 % to 90 %	Good
90 % to 100 %	Excellent

GEOTECHNICAL DESCRIPTION

Geotechnical description, including strata boundaries, is an interpretation of the processed data available at the time of the preparation of a geotechnical log. Subsequent processing and integration of supplementary ground investigation data can allow improved accuracy or confidence. Supplementary information can include:

- geological information
- 2D/ 3D geophysical data
- results of nearby geotechnical investigation locations
- borehole geophysical data
- in situ test data
- laboratory test results
- analysis of drilling parameters such as torque, feed, drill fluid pressure and drilling time.

Any graphical presentation of test results considers values within the scale limits only. No automatic scaling applies, unless indicated otherwise.

GRAPHIC LOG

A geotechnical log can include a graphic log of ground conditions. Figures 2 through 4 present details for soils, cementation degrees and rocks.

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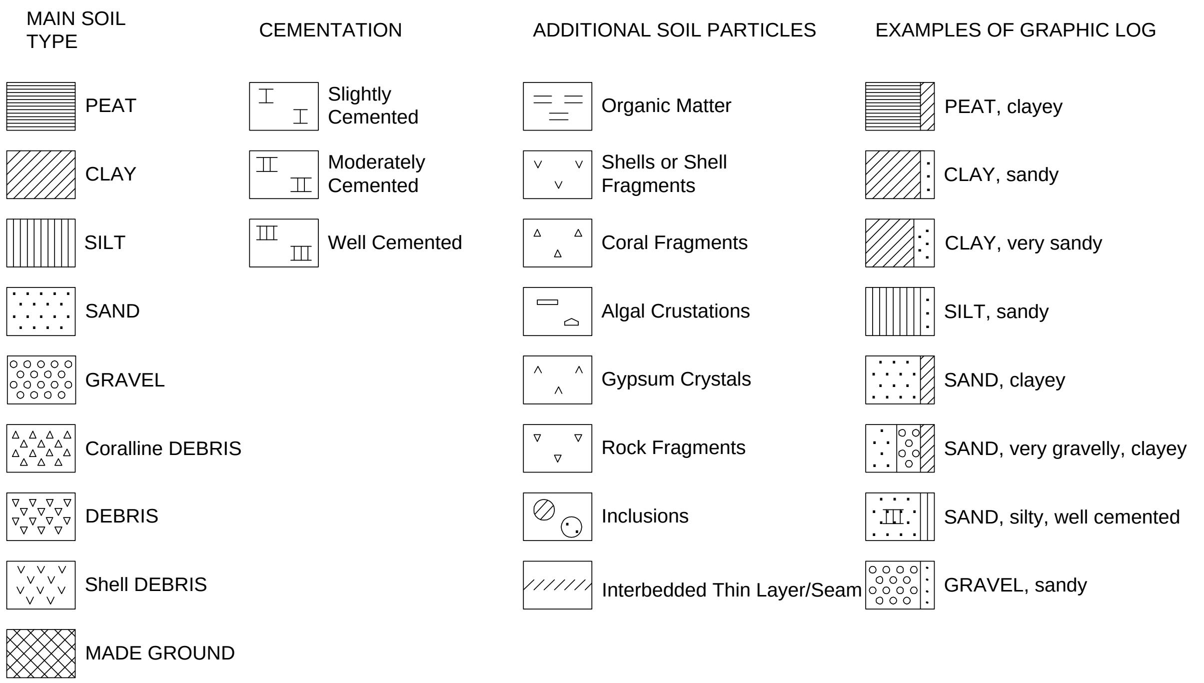

Figure 2 - Symbols for soils

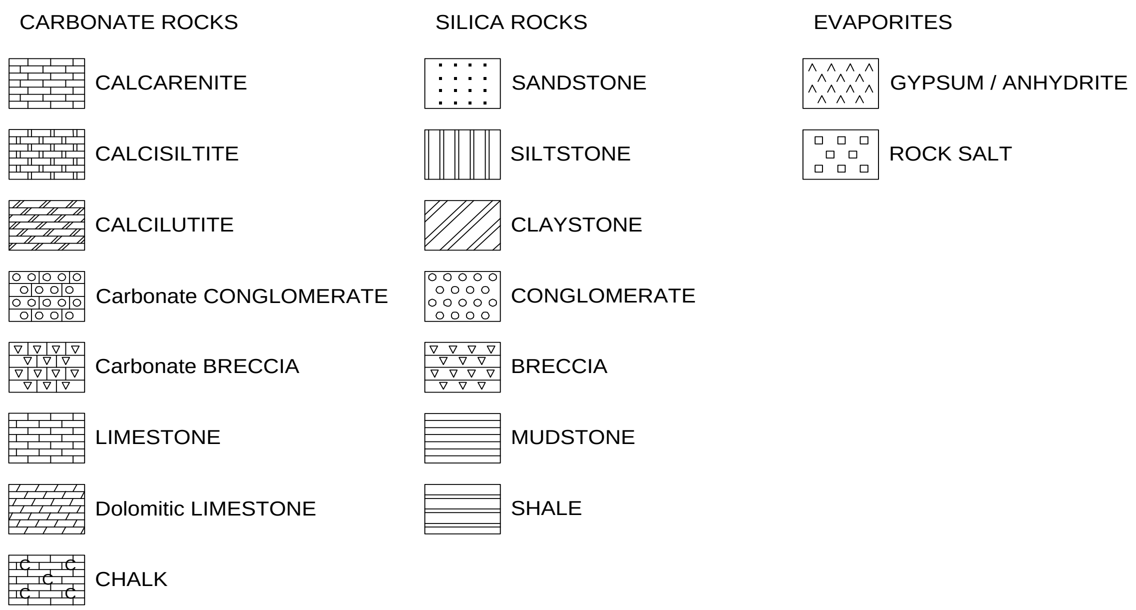

Figure 3 - Symbols for sedimentary rocks

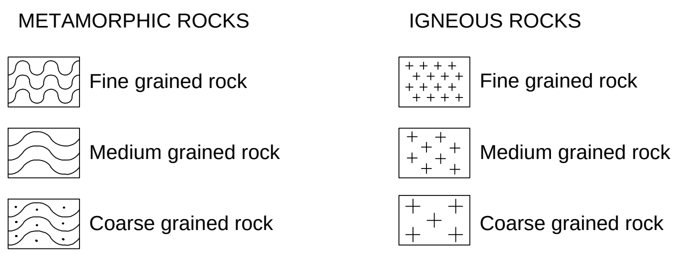

Figure 4 - Symbols for metamorphic and igneous rocks

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WATER LEVEL

Water level measurements taken in boreholes can be valuable. Interpretation of water levels requires due caution. They may or may not be representative of the ground water levels. In any case, water levels apply to the time and date of the measurements only. They will vary due to seasonal and other environmental influences, including construction activities.

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INTRODUCTION

The cone penetration test (CPT) involves the measurement of the resistance of ground to steady and continuous penetration of a cone penetrometer equipped with internal sensors. The measurements comprise penetration depth, cone resistance, sleeve friction and, optionally, pore pressure and inclination from vertical. These measurements permit interpretation of ground conditions.

CPT apparatus and procedures adopted by Fugro are in general accordance ISSMGE (1999), ASTM (2012), ISO (2012) and ISO (2014). BS 5930 (BSI, 2015) refer to ISSMGE (1999). General agreement also applies to Eurocode 7 (CEN, 2007).

Fugro offers CPT systems operated from (1) ground surface and seafloor (non-drilling deployment mode) and (2) downhole in a borehole (drilling deployment mode).

CPT APPARATUS

GENERAL

CPT apparatus includes various parts as described below:
- Thrust machine: apparatus providing thrust to the push rods so that the recommended rate of penetration (20 mm/s) is controlled;
- Reaction equipment: reaction for the thrust machine;
- Push rod: thick-walled cylindrical tube used for advancing the penetrometer to the required test depth. Push rods may also consist of drill pipe;
- Friction-cone penetrometer (CPT): cylindrical terminal body mounted on the lower end of the push rods, including a cone, a friction sleeve and internal sensing devices for the measurement of cone resistance, sleeve friction and, optionally, inclination;
- Piezocone penetrometer (CPTU or PCPT): cylindrical terminal body mounted on the lower end of the push rods, including a cone, a friction sleeve, a filter and internal sensing devices for the measurement of cone resistance, sleeve friction, pressure and, optionally, inclination;
- Measuring system: apparatus and software, including sensors, data transmission apparatus, recording apparatus and data processing apparatus.

DEPLOYMENT FROM GROUND SURFACE OR SEAFLOOR

Specific additional apparatus for CPT deployment from ground surface and seafloor can include:
- Push rod casing: guide for the part of the push rods protruding above the soil, and for the push rod length exposed in water or soil, in order to prevent buckling when the required penetration pressure increases beyond the safe limit for the exposed upstanding length of push rods;
- Friction reducer: ring or special projections fixed on the outside of the push rods, with an outside diameter larger than the base of the cone, to reduce soil friction acting on the push rods.

DOWNHOLE BOREHOLE DEPLOYMENT

Downhole CPT systems latch into a bottom hole assembly at the lower end of a drill pipe. System options are:
1. Operation of a downhole thrust machine by applying mud pressure in the borehole;
2. Remote control of a downhole thrust machine by hydraulic pressure transmitted through an umbilical cable connected to a surface-based pump unit, together with;
3. Application of thrust to drill rods where CPT apparatus and a short push rod are latched in the bottom hole assembly; the thrust machine is at ground surface or seafloor.

Data recording can be surface-based and/or downhole.

Downhole CPTs require drilling apparatus for advancing the borehole. The maximum CPT stroke is generally 1.5 m or 3 m.

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CONE PENETROMETER

Typical features of Fugro penetrometers (Figure 1) include:

- Cone base areas of 500 mm^2, 1000 mm^2 or 1500 mm^2; other sizes are also in use, e.g. 3300 mm^2;
- Cone and friction sleeve sensors placed in series, i.e. subtraction-type penetrometers;
- Pore pressure measurements either at the face of the cone (u_1) or at the cylindrical extension of the cone (u_2). Multiple-sensor penetrometers (u_1, u_2 and u_3) are also available. The u_3 location is immediately above the friction sleeve;
- Inclinometer;
- Storage of signals from the penetrometer in digital form for subsequent computer-based processing and presentation.

PROCEDURE

Figure 2 summarises the test procedure. The procedure includes several stages. The stage of Additional Measurements is optional.

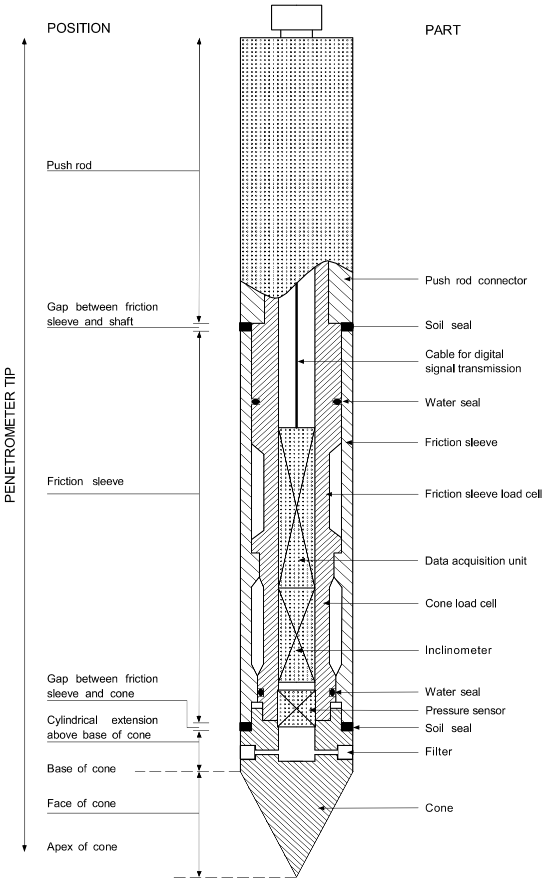

Figure 1 – Piezocone Penetrometer

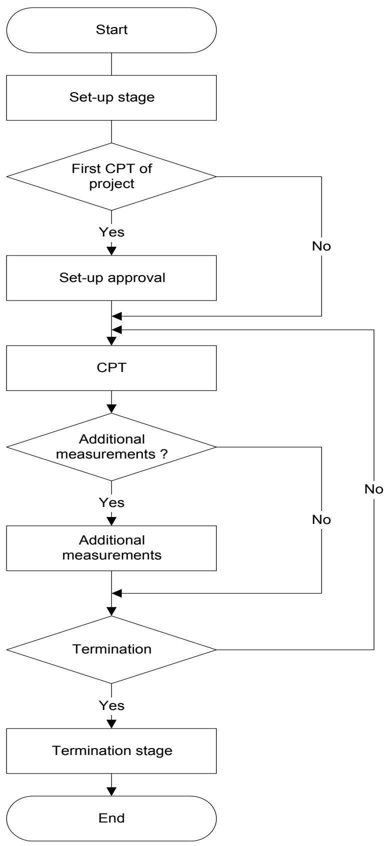

Figure 2 - Flow chart

Set-up requires a reasonably flat, accessible, ground surface with a slope of 5^o or less. In other cases, set-up is at discretion of the equipment operator, considering risks such as damage to apparatus or safety of personnel. Most onshore thrust machines have levelling facilities allowing a vertical start of penetration. Seabed frames used for offshore CPT activities have no levelling facilities, i.e. start of penetration may not be vertical.

The set-up stage includes selection of equipment and procedures according to project-specific agreements, such as a required accuracy class or application class, penetration, type of cone penetrometer and data processing/ submission. Table 1 presents ISSMGE accuracy classes and Tables 2 and 3 summarise ISO application classes. The allowable minimum accuracy of a measured parameter is the larger value of the two quoted. A percentage value applies to the measured value and not to the measuring range. The

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concept of application classes considers intended soil conditions for selection of an application class. For example, Application Class 1 of ISO (2014) can be selected for “very soft to soft soil deposits”, which is approximately equivalent to q_c < 0.5 to q_c < 1 MPa. In other words, Application Class 1 should not apply to “mixed bedded soil profiles with weak to strong layers”.

The accuracy values apply to ground surface or seafloor as reference. They are uncoupled from uncertainty of spatial position below ground surface or seafloor.

Table 1 Accuracy Classes (ISSMGE, 1999)

Accuracy Class	Measured Parameter	Allowable Minimum Accuracy	Maximum Length between Measurements
1	Cone resistance, q_c Sleeve friction, f_s Pore pressure, u Inclination, i Penetration depth, z	50 kPa or 3 % 10 kPa or 10 % 5 kPa or 2 % 2° 0.1 m or 1 %	20 mm
2	Cone resistance, q_c Sleeve friction, f_s Pore pressure, u Inclination, i Penetration depth, z	200 kPa or 3 % 25 kPa or 15 % 25 kPa or 3 % 2° 0.2 m or 2 %	20 mm
3	Cone resistance, q_c Sleeve friction, f_s Pore pressure, u Inclination, i Penetration depth, z	400 kPa or 5 % 50 kPa or 15 % 50 kPa or 5 % 5° 0.2 m or 2 %	50 mm
4	Cone resistance, q_c Sleeve friction, f_s Penetration length, l	500 kPa or 5 % 50 kPa or 20 % 0.1 m or 1 %	50 mm

Table 2 Application Classes (ISO, 2012)

Appl. Class	Measured Parameter	Allowable Minimum Accuracy	Maximum Length between Measurements
1	Cone resistance, q_c Sleeve friction, f_s Pore pressure, u Inclination, i Penetration length, l	35 kPa or 5 % 5 kPa or 10 % 10 kPa or 2 % 2° 0.1 m or 1 %	20 mm
2	Cone resistance, q_c Sleeve friction, f_s Pore pressure, u Inclination, i Penetration length, l	100 kPa or 5 % 15 kPa or 15 % 10 kPa or 3 % 2° 0.1 m or 1 %	20 mm
3	Cone resistance, q_c Sleeve friction, f_s Pore pressure, u Inclination, i Penetration length, l	200 kPa or 5 % 25 kPa or 15 % 50 kPa or 5 % 5° 0.2 m or 2%	50 mm
4	Cone resistance, q_c Sleeve friction, f_s Penetration length, l	500 kPa or 5 % 50 kPa or 20 % 0.2 m or 2 %	50 mm

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Table 3 Application Classes (ISO, 2014)

Application Class	Measured Parameter	Allowable Minimum Accuracy
1	Cone Resistance Sleeve friction Pore pressure	35 kPa or 5 % 5 kPa or 10 % 25 kPa or 5 %
2	Cone resistance Sleeve friction Pore pressure	100 kPa or 5 % 15 kPa or 15 % 50 kPa or 5 %
3	Cone Resistance Sleeve friction Pore pressure	200 kPa or 5 % 25 kPa or 15 % 100 kPa or 5 %

The set-up stage or the termination stage includes the location survey, i.e. the determination of the co-ordinates and the ground surface elevation (or the water depth).

The set-up stage and the termination stage for a downhole CPT include lowering of the CPT apparatus into the borehole and lifting respectively. Most projects require multiple downhole tests in a single borehole.

For piezocone testing, the set-up stage also includes the following steps:
- Office-based or site-based: de-airing of the filter in glycerine by application of 24-hour vacuum and storage in a glycerine-filled container;
- On-site: glycerine filling of hollow space in the cone penetrometer and subsequent mounting of the filter;
- On-site: application of a flexible membrane around the filter to prevent loss of saturating fluid prior to the start of a test.

Land-based tests may include specific measures to help retention of filter saturation during penetration of partially saturated zones. Relaxation of requirements typically applies to offshore tests where water pressures will force entrapped air into solution.

Criteria for test termination are as follows, unless specifically agreed otherwise:
- As instructed by Client;
- Reaching target penetration;
- Reaching maximum capacity of the thrust machine, reaction equipment, push rods and/or measuring sensors;
- Sudden increase in penetrometer inclination;
- Risk of damage to apparatus or safety of personnel, at discretion of equipment operator or as determined by software algorithms;

whichever occurs first and as applicable. Note that ASTM and ISO standards provide no specific requirements for maximum penetrometer inclination from vertical. A value of 15^o is commonly considered.

A CPTU pore pressure measuring system is intended for use in water-saturated uncemented fine-grained soil. Pore pressure measurements (u) are commonly assumed to represent pore water pressures. This assumption is reasonable for soils saturated under in situ stress conditions and remaining saturated during penetration of the cone penetrometer. Results obtained for ground conditions such as partially saturated soils, very dense sands and cemented soils may not be reliable and/or repeatable. For example, stiffness differences between the steel components of the cone penetrometer and the piezocone filter can affect results for very dense sands. Also, loss of saturation of the pore pressure measuring system may occur during a test (Lunne et al. 1997; Peuchen and Terwindt 2014). Reasons can include:
- penetration of partially saturated ground, for example ground containing significant amounts of gas;
- measurement of negative pore pressures such that cavitation occurs. For example, this is not uncommon for a piezocone filter located at the cylindrical extension above the base of the cone (u_2 location). Particularly, this may occur at the time of penetration of dense sand or overconsolidated clay layers. Loss of saturation usually causes a sluggish pore pressure response during penetration of ground below the zone causing desaturation of the pore pressure measuring system.

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Measured pore pressures affected by desaturation of the pore pressure measurement system may not be representative of soil behaviour. Consequently, derived parameters that use pore pressure may also not be representative. Re-saturation of a pore pressure measurement system can take place upon further penetration into soil. In this regard it should be noted that pressure u_2 at the cylindrical extension is commonly assumed equal to u_{2g} in the gap. The assumption $u_2 = u_{2g}$ is probably reasonable for marine conditions, saturated low-permeability soils (clays) that are normally consolidated or lightly overconsolidated and where the gap can become saturated by adequate supply of water and/or water pressure. A similar comment applies to u_3.

Special apparatus and procedures may apply to:

- Specific additional measurements (for example shear wave velocity);
- Specific applications (for example deepwater tests or measurements for application (accuracy) Classes 1 and 2).

RESULTS

CPT PARAMETERS

Presentation of results from cone penetration tests typically includes:

- CPT parameters q_c, f_s and R_f versus depth below ground surface or versus elevation;
- Additional CPTU parameters u_1 or u_2 and, optionally, q_t, q_n, B_q, Q_t, Q_{tn}, F_r and I_c for tests with pore pressure measurements;
- Optionally, inclination i for tests with inclination measurements;
- Standard graphical format and optional ASCII and AGS formats.

Most standards specify scales for graphical presentation as follows:

- Axis for penetration depth z: 1 scale unit = 1 m;
- Axis for cone resistance q_c, corrected cone resistance q_t and net cone resistance q_n:
  1 scale unit = 2 MPa or 0.5 MPa;
- Axis for sleeve friction f_s: 1 scale unit = 50 kPa;
- Axis for friction ratio R_f: 1 scale unit = 2 %;
- Axis for pore pressure u: 1 scale unit = 0.2 MPa or 0.02 MPa;
- Axis for pore pressure ratio B_q: 1 scale unit = 0.5.

Graphical presentation aims for these scale units and scale ratios, where suitable and practicable.

This reference level of a test is (1) the ground surface for onshore tests, (2) the seafloor for nearshore and offshore tests. Historically, the bottom of the borehole was used as the reference level of downhole tests. Data processing presumes a hydrostatic pore pressure profile relative to seafloor, unless specifically indicated otherwise. The definition of CPT parameters is as follows:

z = penetration depth relative to ground surface or seafloor, corrected for inclination from vertical (i) where a test includes inclination measurements, as follows:

$$z = \int_0^l \cos i \,.\, dl$$

where:
z = penetration depth for the conical base of the cone penetrometer
l = recorded penetration length
i = recorded inclination from vertical

q_c = cone resistance relative to the reference level of the test.

f_s = sleeve friction relative to the reference level of the test. A calculated depth correction applies so that the presented sleeve friction corresponds with the cone depth.

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f_t = corrected sleeve friction relative to the reference level of the test. Sleeve friction is corrected for pore pressures acting on the end areas of the friction sleeve

$$f_t = f_s - \frac{(u_2 * A_{sb} - u_3 * A_{st})}{A_s}$$

or simplified to:

$$f_t = f_s - u_2 \frac{(A_{sb} - A_{st})}{A_s} \quad \text{or}$$

$$f_t = f_s - (u_2 * a_{fs})$$

where:
A_{sb} = cross sectional area in the gap between the friction sleeve and the cone
A_{st} = cross sectional area in the gap above the friction sleeve
A_s = surface area of the friction sleeve
a_{fs} = net area ratio of the friction sleeve $(A_{sb} - A_{st})/A_s$

R_f = ratio of sleeve friction to cone resistance (f_s/q_c). This calculated ratio is for the cone depth.

R_{ft} = corrected friction ratio (f_s/q_t). The ratio f_t/q_t applies if f_t is known.

I_{SBT} = non-normalized soil behaviour type index (Robertson, 2010)

$$I_{SBT} = [(3.47 - \log(q_c/P_a))^2 + (\log R_f + 1.22)^2]^{0.5}$$

where:
P_a = atmospheric pressure

u_1 = pore pressure at the face of the cone, relative to the reference level of the test.

u_2 = pore pressure at the cylindrical extension above the base of the cone or in the gap between the friction sleeve and the cone, relative to the reference level of the test.

u_3 = pore pressure immediately above the friction sleeve or in the gap above the friction sleeve, relative to the reference level of the test.

q_t = corrected cone resistance (also called total cone resistance). This includes corrections for hydrostatic and transient pore pressures, and cone construction. The corrected cone resistance is relative to ground surface or seafloor:

Ground surface / seafloor:
$q_t = q_c+(1-a)u_2$ or
$q_t = q_c+(1-a)\{K(u_1-u_o)+u_o\}$

Downhole (historic):
$q_t = q_c+(1-a)u_2+u_{oi}$ or
$q_t = q_c+(1-a)\{K(u_1+u_{oi}-u_o)+u_o\}+au_{oi}$

where:

a = net area ratio of the cross-sectional steel area at the gap between cone and friction sleeve to the cone base area. This ratio is penetrometer-type dependent. The a-factor indicates the effect of pore pressure on unequal cross-sectional areas of the cone.

u_o = hydrostatic pore pressure at the cone, relative to the phreatic surface or the seafloor. This is a calculated value.

u_{oi} = hydrostatic pore pressure at the bottom of the borehole, relative to seafloor. This is a calculated value.

K = adjustment factor for the ratio of pore pressure at the cylindrical extension above the base of the cone to pore pressure on the cone face.

Ground surface / seafloor:
$K = (u_2-u_o)/(u_1-u_o)$

Downhole (historic):
$K = (u_2+u_{oi}-u_o)/(u_1+u_{oi}-u_o)$

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The term u_2-u_o or u_2+u_{oi}-u_o refers to excess pore pressure (with respect to hydrostatic pore pressure). Common symbols for excess pore pressure are du_2 or Δu_2. Similarly, du_1 or Δu_1 may represent the term u_1-u_o or u_1+u_{oi}-u_o.

The K-factor is only of interest for processing of CPTU results with pore pressure measurement at the cone face (u_1). The factor depends on soil characteristics as fabric, overconsolidation ratio, compressibility and crushability. The K-factor is estimated from:

$$K = 0.91e^{-0.09Q_t^{0.47}}\left(\frac{1}{1+F_r(0.17+0.061(Q_t-21.6)^{1/3})} - e^{-2F_r}\right) \qquad \text{(Peuchen et al., 2010)}$$

q_n = q_t-σ_{vo} = net cone resistance. This includes corrections for hydrostatic and transient pore pressures, in situ stress, and cone construction. The symbol for q_n may also be q_{net}.

where:
σ_{vo} = total in situ vertical stress at the cone base, relative to ground surface or seafloor. This is a calculated value.

Q_t = q_n/σ'_{vo} = normalized cone resistance

where:
σ'_{vo} = effective in situ vertical stress at the cone base, relative to ground surface or seafloor. This is a calculated value.

Q_{tn} = normalized cone resistance with variable stress exponent n, where:

$Q_{tn} = [(q_t - \sigma_{vo})/P_a]\,(P_a/\sigma'_{vo})^n$

$n = 0.381\,(I_c) + 0.05\,(\sigma'_{vo} / P_a) - 0.15$ and $n \le 1$ (Zhang et al., 2002)

I_c = soil behaviour type index (Robertson and Wride,1998)

$I_c = [(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2]^{0.5}$

F_r = f_t/q_n = normalized friction ratio.

B_q = pore pressure ratio.

Ground surface / seafloor:	Downhole (historic):
B_q = $K(u_1-u_o)/q_n$ or	B_q = $K(u_1+u_{oi}-u_o)/q_n$ or
B_q = $(u_2-u_o)/q_n$	B_q = $(u_2+u_{oi}-u_o)/q_n$

Presented values for u_2, q_t, q_n and B_q may be denoted by u_2*, q_t*, q_n*, B_q*, Q_t* and F_r* if u_2 is derived rather than measured, for example if derived by applying a K-factor.

Some deployment systems allow monitoring of CPT parameters in reverse mode, i.e. upon retraction of the cone penetrometer. This optional feature presents additional information that can improve interpretation of ground behaviour, for example strength sensitivity of fine-grained soil.

ACCURACY OF MEASUREMENTS

Accuracy Classes and Application Classes

Cone penetration test standards can follow a “prescriptive” approach, whereby specific detailed measures provided a “deemed to comply” practice. ASTM (2012) is an example of this approach. ISSMGE (1999) and ISO (2012, 2014) specify “performance” criteria for cone penetration test measurements. The ISO standard on metrological confirmation (ISO, 2003) provides the general framework for assessment of performance compliance. Peuchen and Terwindt (2014, 2015) provide guidance on uncertainty estimation for cone penetration test results.

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The following comments apply:

- Accuracy is the “closeness of a measurement to the true value of the quantity being measured”. It is the accuracy as a whole that is ultimately important not the individual parts. Precision is the “closeness of each set of measurements to each other”. The resolution of a measuring system is the “minimum size of the change in the value of a quantity that it can detect”. It will influence the accuracy and precision of a measurement.
- Accuracy Class 3 and Application Class 3 typically represents industry practice. They are approximately equivalent to the more implicit requirements of ASTM International. Class 3 applies, unless specifically agreed otherwise.

So-called “zero drift” of a measured parameter is an approximate performance indicator for the measuring system. Zero drift is the absolute difference of the zero readings, reference readings or zero reference reading of a measuring system between the start and completion of the cone penetration test. The reference readings can be taken at (1) atmospheric pressure at ground surface or above water level or (2) under hydrostatic water pressure close to seafloor. The zero drift of the measured parameters should be within the allowable minimum accuracy according to the selected accuracy class. Correction of measured parameters for zero drift can be adopted if appropriate. Peuchen and Terwindt (2014) provide comments on interpretation of zero drift values.

Accuracy considerations for strongly layered soils should allow for heat flux phenomena. Heat flux gives an apparent shift in cone resistance (Post and Nebbeling, 1995). For example, friction in dense sand causes a cone to heat by about 1^{o}C/MPa cone resistance. Resulting heat flux decreases cone resistance by an apparent shift in the order of 100 kPa to 200 kPa for a penetrating probe going from dense sand into clay. This is a temporary decrease lasting about 5 minutes. Ambient temperature compensation systems cannot avoid heat flux effects. Penetration interruption can serve as mitigation measure.

Shallow Penetration

Use of reaction equipment will affect stress conditions for shallow penetration. Particularly, offshore conditions may include extremely soft ground at seafloor. Soil disturbance, pore pressure build-up and consolidation of near-surface soft soil may take place. This will affect the measurements.

Downhole borehole deployment implies a typical limiting CPT stroke of 1.5 m or 3 m. It is common to perform multiple semi-continuous tests. Graphics for such tests typically show a build-up of CPT values for the initial 0.1 m to 0.5 m penetration. This penetration zone is immediately below the required borehole and represents complex ground stress conditions and/or borehole-induced ground disturbance that cannot be avoided.

Penetration Interruption

A penetration interruption may be unavoidable, for example to add a push rod or to perform a pore pressure dissipation test. This will affect test results.

Consolidation of low-permeability soil around a cone tip is of particular interest. A stationary cone penetrometer can apply local stresses that approach failure conditions, i.e. about 9 times the undrained shear strength or about 2 times the in situ mean effective stress. Pore pressure re-distribution and dissipation occur, resulting in a local increase in undrained shear strength and hence cone (bearing) resistance. A doubling of cone resistance may not be unreasonable for 100 % consolidation. Supplementary considerations include:

- Small downward movement of a penetrometer (order of millimetres) during a test can contribute to maintaining local stresses approaching failure conditions;
- Soil consolidation around a cone penetrometer may lead to soil/penetrometer adhesion that is sufficient to give an increase in “cone” diameter. Resumption of penetration will lead to loss of adhered soil, usually within an equivalent distance of a few times the cone diameter;
- A low B_q value may imply partially drained penetration conditions. It is likely that any steady-state penetration conditions will not apply instantaneously upon resumption of penetration;
- Measuring sensors in a probe generate heat, but this is probably not significant for any stationary measurement. Fugro’s strain-gauge load sensors are compensated for ambient temperature fluctuations.

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Depth Measurement for Offshore Conditions

Offshore definition of the seafloor (ground surface) is difficult for extremely soft ground at seafloor. Penetration of the reaction equipment into a near-fluid zone of the seabed may take place unnoticed. Such settlement affects the start of penetration depth z. Also, settlement may continue at the time of testing.

Downhole CPT systems rely on depth control applicable to borehole drilling. Depth control according to Z2 of Table 4 is feasible for drilling systems deployed from a fixed platform, for example a jack-up. This value excludes uncertainty associated with determination of seafloor level. Drilling control from floating equipment, for example a geotechnical survey vessel, may be subject to the additional influence of waves and tides. Z2 is typically feasible for favourable conditions. Z3 or Z4 may apply for adverse conditions.

Table 4 Depth Accuracy Classes According to ISO (2014)

Depth Accuracy Class	Maximum Data Point Depth Uncertainty [m]
Z1	0.1
Z2	0.5
Z3	1.0
Z4	2.0
Z5	> 2.0

Zero-Correction for Offshore Conditions

Water pressures generate significant values of cone resistance and pore pressure. The standardised practice is to correct these reference readings to zero at seafloor. CPT systems for non-drilling mode and for seafloor drilling mode allow zero-correction to hydrostatic conditions prior to the start of a test, typically with a zero-correction uncertainty approaching the resolution of the CPT system. Downhole borehole CPT systems latch into the lower end of a drill pipe. The pressure conditions in the drill pipe may not be in full equilibrium with the surrounding ground water pressure and zero-correction will be subject to increased uncertainty, i.e. uncertainty for pore pressure in the order of 100 kPa for deepwater tests (Peuchen, 2000). This uncertainty depends on factors such as the free-flow and viscosity of drill fluid between the drill bit and the seafloor. The uncertainty typically decreases with decreasing depth of the drill bit below sea level and below seafloor. Uncertainty for the zero-correction of cone resistance is approximately equivalent, but by a factor representing the net area ratio effect.

Deepwater Tests

A deepwater environment presents some favourable conditions for cone penetration tests, notably temperature. Ambient temperature conditions are practically constant and the measuring system has ample time to adjust to these temperatures. In addition, transient heat flow phenomena in a cone penetrometer are usually not applicable. This is because a cone penetrometer accumulates negligible (frictional) heat when penetrating the generally prevalent soils of very soft consistency.

Deepwater (piezocone) pore pressure measurements are essentially similar to shallow-water measurements, with the exception of an increased measuring range for pore pressure leading to some reduction in sensor accuracy. Saturation of a pore pressure measuring system is excellent for a deepwater environment, as the high pressures will force any gas bubbles into solution.

Currently available evidence indicates that a high-quality subtraction-type cone penetrometer is adequate for very soft soil characterisation to a water depth of 3000 metres and probably beyond.

ADDITIONAL MEASUREMENTS

Friction-cone and piezocone penetrometers allow specific additional measurements, such as friction set-up tests, pore pressure dissipation tests and measurements of ground water pressure. These additional measurements require a penetration interruption or may be feasible at the end of a test. It is also common to add other in situ test devices to a cone penetrometer. Table 5 presents the more common types.

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Table 5 Probes for Additional In Situ Tests

Type of Probe	Properties	Units
Electrical Conductivity Penetrometer (ECPT)	Electrical conductivity, K	S/m
Temperature Cone Penetrometer (TCPT)	Temperature, T	°C
Seismic Cone Penetrometer (SCPT)	Shear wave velocity, v_s	m/s
Cone Pressuremeter (CPMT)	Shear stress-strain-time response, σ, ε, t	MPa, -, s
Natural Gamma Penetrometer (GCPT)	Natural gamma ray, γ	CPS
Cone Magnetometer (CMMT)	Magnetic flux density B, magnetic field horizontal angle θ and vertical angle ϕ	μT, °, °
Hydraulic Profiling Tool (HPT)	Permeability, k	m/s
S = Siemens m = metre s = second	Pa = Pascal CPS = counts per second T = Tesla	

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INTRODUCTION

This document presents a summary of interpretation methods for cone penetration test (CPT) results. The project-specific selection of methods depends on the agreed project requirements. Some of the methods suit computer-based interpretation of CPT data records.

Interpretation of cone penetration test results helps provide parameters for geotechnical models. Conventional models are typically based on plasticity theory for ultimate limit states, and on elasticity theory and consolidation theory for serviceability limit states. Features of these geotechnical models are:

– analysis of either drained (sand model) behaviour or undrained (clay model) behaviour for plasticity models
– analysis for the ultimate limit state differs from that for the serviceability limit state.

CPT interpretation methods are mostly based on empirical correlations with limited theoretical backing. Data integration with other, complementary investigation techniques (such as drilling, sampling and laboratory testing) improves confidence levels.

The interpretation techniques discussed below are subject to limitations such as:

– The majority of interpretation methods apply to "conventional" sands and clays. Conventional methods may not be appropriate for silts, sand/clay/gravel mixtures, varved or layered soils, gassy soils, underconsolidated soils, peats, carbonate soils, cemented soils and residual soils. These non-conventional soils warrant a more specific approach.
– Empirical correlations use reference parameters such as the undrained shear strength determined from a laboratory single-stage Isotropically Consolidated Undrained triaxial test (CIU) on an undisturbed specimen obtained by means of push sampling techniques (Van der Wal et al., 2010). The reference parameter may not be appropriate for the selected geotechnical model, and adjustment may be necessary. Also, adjustment for test conditions may be necessary, for example in situ temperature versus laboratory temperature.
– The cone penetration test offers limited direct information on serviceability limit states (deformation), as the penetration process imposes large strains in the surrounding soil. In comparison to ultimate limit states, better complementary data will usually be required.
– CPT interpretation techniques are often indirect. Usually, interpretation requires estimates of various other parameters. This is consistent with an integrated geotechnical investigation approach. Inevitably, this approach also includes some redundancy of data.
– Drained or undrained behaviour for the geotechnical analysis at hand may or may not coincide with respectively drained or undrained behaviour during fixed-rate penetration testing. This interpretation difficulty remains largely unresolved at this time.
– The interpretations apply to conditions as encountered at the time of the geotechnical investigation. Geological, environmental and construction/operational factors may alter as-found conditions.

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PENETRATION BEHAVIOUR

Soil behaviour during cone penetration testing shows large displacements in the immediate vicinity of the penetrometer, and small elastic displacements further away from the penetrometer. Density/structure, stiffness and in situ stress conditions significantly affect the measured parameters.

The measured cone resistance (q_c) includes hydrostatic water pressures as well as stress-induced pore pressures. The pore pressures are usually negligible for clean sand because the ratio of effective stress to pore pressure is high. This ratio is, however, low for penetration into clay. Knowledge of pore pressures around the penetrometer can thus be important. CPT parameters that take account of pore pressure effects include total cone resistance (q_t), net cone resistance (q_n) and pore pressure ratio (B_q). These parameters can be calculated if Piezo-cone Penetration Test (PCPT or CPTU) data are available. The influence of pore pressures on sleeve friction f_s is relatively small. It is common to ignore this influence. Calculation of friction ratio R_f (defined as f_s/q_c) includes no allowance for pore pressure effects.

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The penetration rate with respect to soil permeability determines whether soil behaviour is primarily undrained, drained or partially drained. Partial drainage may also be denoted as partial consolidation. In general, soil behaviour during cone penetration testing is drained in clean sand (no measurable pore pressures as a consequence of soil displacements) and undrained in clay (significant pore pressure changes). Partially drained behaviour occurs in soils with intermediate permeability, such as sandy silt. Results of a pore pressure dissipation test can provide indications for partial drainage conditions.

Particularly, partial drainage conditions should be considered when t_{50} is less than about 100 s (DeJong and Randolph, 2012). The term t_{50} represents the time for 50 % dissipation of excess pore pressure at the u_2 location of a cone penetrometer.

The following sections mostly consider interpretation of drained soil behaviour (sand) and undrained soil behaviour (clay).

SOIL BEHAVIOUR IDENTIFICATION

Identification of soil stratigraphy in terms of general soil behaviour (and to a lesser degree soil type) is a more important feature of CPT than other investigation technique.

Figures 1 to 3 show soil behaviour identification according to procedures given by Robertson (2009) and Ramsey (2002). Robertson (2009) represents an update of Robertson (1990), by exchange of Q_t with Q_{tn}. The procedures consider a normalised soil behaviour classification that provides general guidance on likely soil type (silty sand for example) and a preliminary indication of parameters such as angle of internal friction φ', overconsolidation ratio (OCR) and clay sensitivity (S_t). The procedures require piezo-cone test data:

$$Q_{tn} = [(q_t - \sigma_{vo})/P_a]\,(P_a/\sigma'_{vo})^n \qquad Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \qquad F_r \text{ or } nR_f = \frac{f_s}{q_t - \sigma_{vo}} 100\% \qquad B_q = \frac{u - u_0}{q_t - \sigma_{vo}}$$

where:

Q_{tn}	= normalised cone resistance with variable stress exponent
Q_t	= normalised cone resistance
q_t	= corrected cone resistance
σ_{vo}	= total in situ vertical stress
σ'_{vo}	= effective in situ vertical stress
P_a	= atmospheric pressure
n	= stress exponent
f_s	= measured sleeve friction
u	= measured pore pressure
u_0	= theoretical hydrostatic pore pressure.

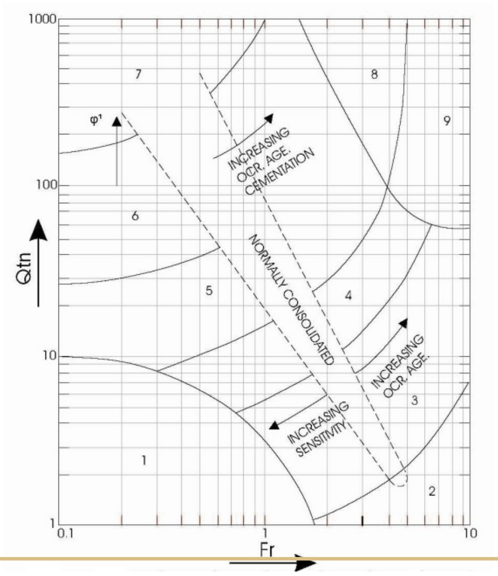
Zhang et al. (2002) defined stress exponent n as follows:

$$n = 0.381\,(I_c) + 0.05\,(\sigma'_{vo} / P_a) - 0.15 \text{ where } n \le 1$$

Robertson and Wride (1998) defined soil behaviour type index I_c (Figure 3) as follows:

$$I_c = [(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2\,]^{0.5}$$

Soils with $I_c < 2.5$ are generally cohesionless, coarse grained, where cone penetration is generally drained and soils with $I_c > 2.7$ are generally cohesive, fine grained, where cone penetration is generally undrained (Robertson, 1990). Cone penetration in soils with $2.5 < I_c < 2.7$ is often partially drained.



1. Sensitive, fine grained
2. Organic soils - peats
3. Clays- clay to silty clay
4. Silt mixtures – clayey silt to silty clay
5. Sand mixtures – silty sand to sandy silt
6. Sands – clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

(*) Heavily overconsolidated or cemented

Figure 1, Classification chart Robertson (2009)

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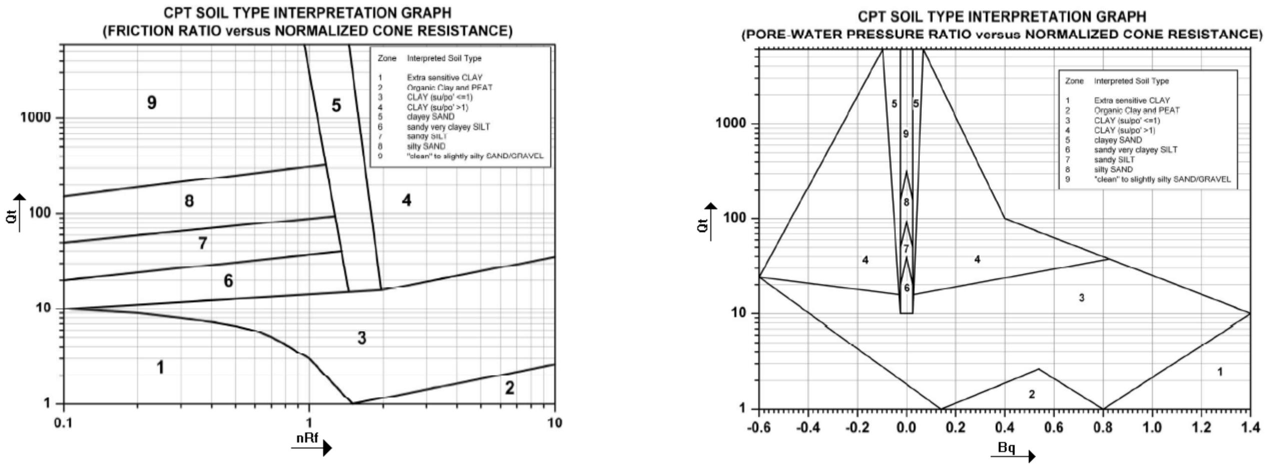

1. Extra sensitive clay
2. Organic clay and peat
3. Clay (s_u/σ'_{vo} <= 1)
4. Clay (s_u/σ'_{vo} > 1)
5. Clayey sand
6. Sandy very clayey silt
7. Sandy silt
8. Silty sand
9. "Clean" sand/gravel

Figure 2, Classification charts Ramsey (2002)

Classification is only possible for certain combinations of Q_{tn}, Q_t , F_r, nR_f and B_q, as shown below.

Classification Limits	
Robertson	**Ramsey**
$1 \leq Q_{tn} \leq 1000$ $0.1 \leq F_r \leq 10$ $-0.2 \leq B_q \leq 1.4$	$1 \leq Q_t \leq 6000$ $0.1 \leq nR_f \leq 10$ $-0.6 \leq B_q \leq 1.4$

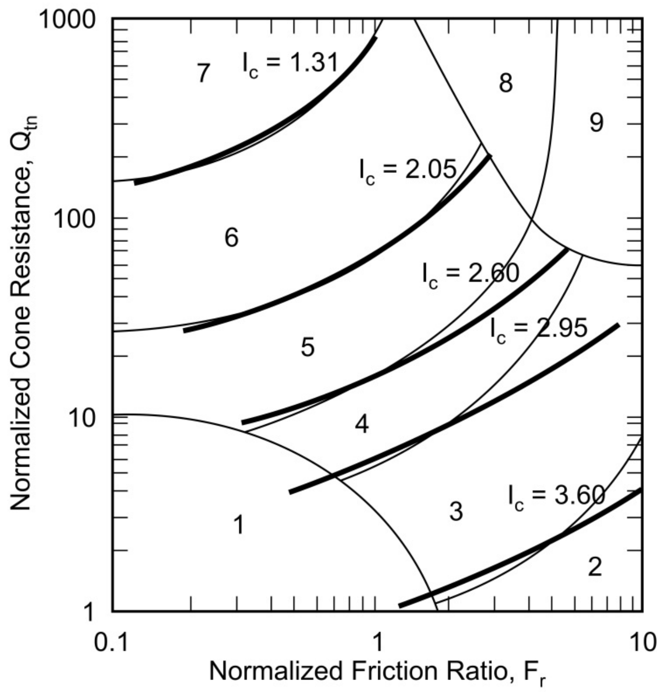

Figure 3, Soil behaviour type index I_c superimposed on Robertson (2009) classification chart

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Figure 4 presents a classification chart for friction cone data according to Robertson (2010). This procedure requires no pore pressure input. A non-normalised soil behaviour type index, I_{SBT} applies:

$$I_{SBT} = [(3.47 - \log(q_c/P_a))^2 + (\log R_f + 1.22)^2]^{0.5}$$

I_{SBT} is similar to I_c. Values for I_{SBT} and I_c are typically comparable for effective in situ vertical stress between 50 kPa and 150 kPa.

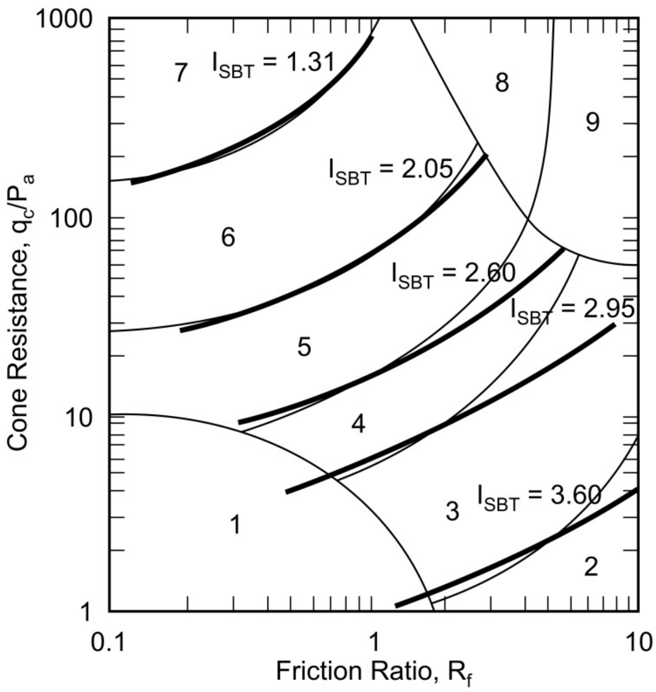

Figure 4, Robertson (2010) classification chart including I_{SBT}

SAND MODEL

Unit Weight – Sand

Unit weight of uncemented (silica) sand, silt and clay soils may be derived according to Mayne et al. (2010):

$$\gamma = 1.95\,\gamma_w \left(\frac{\sigma'_{vo}}{P_a}\right)^{0.06} \left(\frac{f_t}{P_a}\right)^{0.06}$$

where total unit weight γ and unit weight of water γ_w are in kN/m^3 and effective in situ vertical stress σ'_{vo} is in kPa. The symbol f_t refers to sleeve friction corrected for pore pressures acting on the end areas of the friction sleeve, with units in kPa. Atmospheric pressure P_a is in kPa.

In Situ Stress Conditions - Sand

A knowledge of in situ stress conditions is required for estimation of parameters such as relative density D_r and angle of internal friction of a sand deposit φ'. The effective in situ vertical stress σ'_{vo} may be calculated with a reasonable degree of accuracy but the effective in situ horizontal stress $\sigma'_{ho} = K_o\sigma'_{vo}$ is generally unknown. Usually, it is necessary to consider a range of conditions for K_o (coefficient of earth pressure at rest). The range considers overconsolidation as inferred from a geological assessment, pre-consolidation pressures of intermediate clay layers and/or theoretical limits of K_o.

Geological factors concerning overconsolidation include ice loading, soil loading and groundwater fluctuations. Possible subdivisions for these factors are mechanical, cyclic and ageing consolidation.

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K_o may be directly correlated to overconsolidation ratio (OCR), as follows:

$$K_o = 0.4 \sqrt{(OCR)}$$

Mayne and Kulhawy (1982) investigated mechanical overconsolidation of reconstituted laboratory specimens for over 170 different soils. A K_0 OCR correlation requiring effective angle of internal friction as input was found to provide a reasonable match. It can be shown that the $K_o = 0.4 \sqrt{(OCR)}$ equation provides similar statistics to the Mayne and Kulhawy correlation.

No laboratory study can fully capture in situ behaviour. Particularly, K_o may be underestimated if effects such as ageing and cyclic loading are relevant.

In general, in situ K_o values are limited to the range $K_o = 0.5$ to $K_o = 1.5$. For many situations, K_o values are believed to be relatively low at greater depths (say $K_o < 1$ for depths exceeding 50 m). Jamiolkowski et al. (2003) recommend using a limiting value $K_o = 1$ in practice.

Relative Density - Sand

Procedures for estimation of in situ density condition (loose, dense, etc.) consist of:
(a) Estimation of in situ stress conditions σ'_{vo} and $\sigma'_{ho;}$
(b) Empirical correlation of relative density D_r (or density condition) with q_c, σ'_{vo} and σ'_{ho}.

Estimation of stress conditions has been discussed above.

Common relationships between q_c and D_r are based on Cone Penetration Tests carried out in sand samples reconstituted in laboratory calibration chamber tests. Such tests are carried out as part of general geotechnical research projects and are subject to a number of limitations, such as:
– soil type dependence
– inaccuracies in determination of laboratory D_r
– limited range of stress levels and K_o values
– sample preparation and soil stress history simplifications.

Jamiolkowski et al. (2003) proposes the following relationship between q_c and D_r for normally and overconsolidated silica (dry) sands:

$$D_{r(dry)} = \frac{1}{2.96} \ln \left[\frac{\frac{q_c}{P_a}}{24.94 \left(\frac{\sigma'_{vo} \left(\frac{1+2K_o}{3} \right)}{P_a} \right)^{0.46}} \right]$$ and for saturated sands: $$D_{r(sat)} = \left(\frac{-1.87 + 2.32 \ln \frac{q_c}{(P_a * \sigma'_{vo})^{0.5}}}{100} + 1 \right) \frac{D_r(dry)}{100}$$

where relative density D_r is a fraction. The correlation for saturated sands results in relative densities that can be up to about 10% higher compared to the correlation for dry sands.

Determination of laboratory minimum and maximum index dry unit weights (γ_{dmin} and γ_{dmax}) forms the basis for the relative density concept (loose, dense sand, etc.). As yet, there is no internationally agreed procedure. Hence, laboratory test procedure dependence applies. Also, it is unlikely that any of the procedures consistently provide the "lowest" γ_{dmin} or the "highest" γ_{dmax}. In situ soil unit weights may therefore fall outside laboratory ranges. The relative density concept is necessary to provide a link between field investigations and laboratory testing on reconstituted specimens, as undisturbed sampling of sands is expensive.

Calibration chamber test results apply to a limited range of stress conditions only; typically:

$$50 \text{ kPa} < \sigma'_{vo} < 400 \text{ kPa}$$
$$0.4 < K_o < 1.5$$

Sample preparation for laboratory chamber tests is usually by means of dry pluviation. Soil stress history application is by mechanical overconsolidation.

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Angle of Internal Friction - Sand

The effective shear strength parameter φ' is not a true constant. It depends on factors such as density, stress level, shearing mode and mineralogy. There is evidence that overconsolidation ratio, method of deposition and in situ stress anisotropy is less important.

Correlation of angle of internal friction φ' to cone resistance q_c may be done at various levels of sophistication. Simple procedures rely on a conservative assessment of soil behaviour classification. A more sophisticated empirical correlation consists of:

(a) Estimation of in situ stress conditions σ'_{vo} and σ'_{ho}
(b) Estimation of relative density D_r
(c) Empirical correlation of angle of internal friction φ' with D_r, σ'_{vo} and σ'_{ho}.

Estimation of stress conditions and relative density has been discussed above.

The empirical procedure proposed by Bolton (1986 and 1987) is used for estimation of φ'. This correlation applies to clean sands and considers peak secant angle of internal friction in Isotropically Consolidated Drained triaxial compression (CID) of reconstituted sand. This procedure requires estimation of the dilatancy index and the critical state angle of internal friction.

Kulhawy and Mayne (1990) determined an equation based upon 20 data sets obtained from calibration chamber tests. This equation is almost identical to the empirical formula determined earlier by Trofimenkov (1974) which was based on mechanical cone data. Mayne (2007) validated the use of total cone resistance q_t instead of cone resistance q_c used in the equation from Kulhawy and Mayne (1990).

$$\varphi' = 17.6 + 11.0 \log\left(\left(\frac{q_t}{P_a}\right) / \left(\frac{\sigma'_{vo}}{P_a}\right)^{0.5}\right) \qquad \text{(Mayne, 2007)}$$

Undrained Shear Strength - Sand

Undrained shear strength of cohesionless soil can be important for assessment of cyclic mobility and liquefaction potential. Geotechnical procedures other than the conventional limit state models are employed.

Compressibility - Sand

Correlations between CPT data and compressibility parameters are indicative only. Further developments in interpretation techniques may offer improvement in the future.

Elasticity theory is commonly employed for analysis of drained soil deformation behaviour. Secant moduli are adopted. A common guideline is an empirical correlation given by Baldi et al. (1989). The correlation is for silica-based sand and considers cone resistance q_c, in situ stress conditions and secant Young's modulus for drained stress change E'. The ratio of E'/q_c typically ranges from about 3 to 5 for recently deposited normally consolidated sands up to about E'/q_c = 6 to 25 for overconsolidated sands. The correlation has been inferred from laboratory conditions; including CPT tests in a calibration chamber and conventional triaxial compression tests on reconstituted sand samples. It takes account of the degree of deformation and overconsolidation. In this regard, it is noted that secant deformation moduli are strongly dependent on strain level: the elastic modulus increases with decreasing strain to an upper limit at about 10^{-4} % strain.

For estimation of initial (small strain) or dynamic shear moduli, ratios of G_{max}/q_c of between about 4 and 20 are considered, in accordance with Baldi et al. (1989).The basis for this correlation is similar to that of secant Young's modulus, except that laboratory resonant column tests serve as reference instead of triaxial compression tests. Results of limited in situ seismic cross-hole and downhole tests provide an approximate check of this correlation.

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Constrained Modulus M - Sand

Kulhawy and Mayne (1990) derived two formulas for the determination of the constrained modulus for both normally consolidated and overconsolidated sands by indicating that the modulus is a function of relative density. The determination of relative density can be done with, for example, the methods indicated previously.

$$M = q_c * 10^{1.09-0.0075D_r} \qquad \text{(Normally consolidated sands, Kulhawy and Mayne, 1990)}$$

$$M = q_c * 10^{1.78-0.0122D_r} \qquad \text{(Overconsolidated sands, Kulhawy and Mayne, 1990)}$$

where D_r is in %, and q_c and M in kPa respectively.

Shear Wave Velocity v_s – Sand

If no in situ measurements of shear wave velocities (v_s) are available, then empirical correlation with CPT parameters may be considered. Hegazy and Mayne (2006) published a statistical correlation derived from 73 sites worldwide representing a range of soil types including sands, clays, soil mixtures and mine tailings (Figure 5). The correlation considers a normalized cone resistance (q_{c1N_hm}) and a soil behaviour type index (I_{c_hm}) as follows:

$$v_s = 0.0831 q_{c1N\_hm} (\sigma'_{vo} / P_a)^{0.25} \; e^{(1.786\, I_{c\_hm})} \qquad \text{(Hegazy and Mayne, 2006)}$$

where shear wave velocity v_s is in m/s and q_{c1N_hm} and I_{c_hm} are dimensionless. Calculations for q_{c1N_hm} and I_{c_hm} require iteration, and consider measured cone resistance q_c or corrected cone resistance q_t, measured sleeve friction f_s, total in situ vertical stress σ_{vo}, effective in situ vertical stress σ'_{vo} and atmospheric pressure P_a.

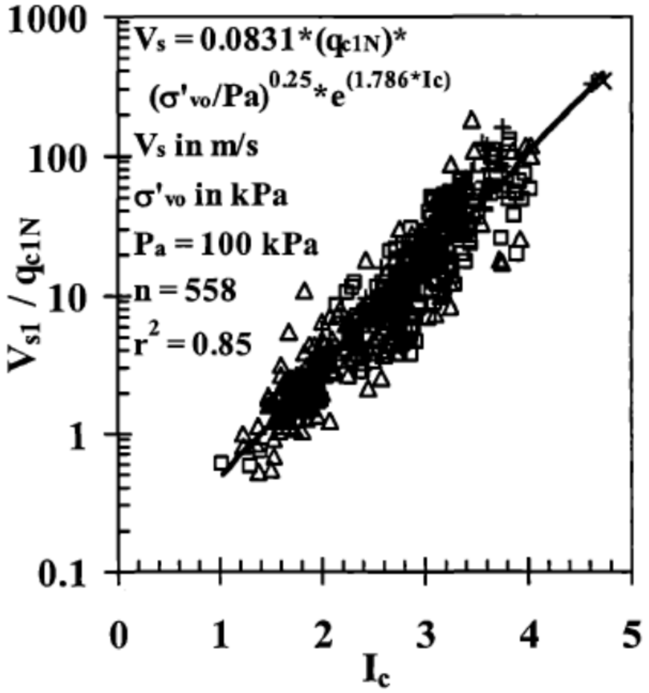

Figure 5, v_s – q_c correlation according to Hegazy and Mayne (2006)

Robertson and Cabal (2010) present a v_s correlation incorporating net cone resistance q_n (= $q_t - \sigma_{vo}$) and soil behaviour type index (I_c) as defined by Robertson and Wride (1998):

$$v_s = [\alpha_{vs}(q_t - \sigma_{vo}) / P_a]^{0.5} \text{ where } \alpha_{vs} = 10^{(0.55\, I_c + 1.68)} \qquad \text{(Robertson and Cabal, 2010)}$$

where shear wave velocity v_s is in m/s and total cone resistance q_t, total in situ vertical stress σ_{vo} and atmospheric pressure P_a are in kPa. The method can be applied to a wide range of soil behaviour types, notably uncemented Holocene to Pleistocene age soils. Older deposits could have a higher shear wave velocity. Exceptions are Zones 1, 8 and 9 of Robertson (1990 and 2009).

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Baldi et al. (1989) derived a correlation between shear wave velocity v_s and cone resistance q_c for uncemented silica sands. This correlation is based on data from CPT, cross-hole and Seismic Cone Penetration Tests (SCPT) performed in quaternary deposits of the predominantly silica Po river sand and Gioia Tauro sand with gravel.

$$v_s = 277 q_c^{0.13} \sigma'_{vo}{}^{0.27} \qquad \text{(Baldi et al., 1989)}$$

where shear wave velocity v_s is in m/s and cone resistance q_c and effective in situ vertical stress σ'_{vo} are in MPa.

Shear wave velocity may be normalised according to Robertson and Cabal (2010):

$$v_{s1} = v_s \cdot (P_a / \sigma'_{vo})^{0.25} \qquad \text{(Robertson and Cabal, 2010)}$$

Shear Modulus G_{max} - Sand

Interpretation of low-strain shear modulus can be considered by using the modified correlation proposed by Rix and Stokoe (1991) in which data from calibration test measurements is compared to the correlation obtained between G_{max} and q_c by Baldi et al. (1989).

$$G_{max} = 1634 (q_c)^{0.25} (\sigma'_{vo})^{0.375} \qquad \text{(Rix and Stokoe, 1991)}$$

where $G_{max,}$ q_c and σ'_{vo} are in kPa.

CLAY MODEL

Unit Weight – Clay

Empirical correlation between unit weight of clay and CPT parameters is as described in "Unit Weight – Sand" above.

In Situ Stress Conditions - Clay

Similar to sand, a knowledge of in situ stress conditions is generally necessary for estimation of other parameters such as consistency (soft, stiff, etc.) of a clay deposit and compressibility.

Calculation of the effective in situ vertical stress σ'_{vo} is reasonably accurate. A more approximate estimate applies to the effective in situ horizontal stress σ'_{ho}, or, more particular, K_o as $\sigma'_{ho} = K_o \sigma'_{vo}$.

Direct correlations for interpretation of the coefficient of earth pressure at rest K_o are uncommon.

For normally consolidated clays and silts, K_{onc} may be correlated with angle of internal friction, in accordance with Jaky (1944), or more simply in accordance with Mayne and Kulhawy (1982). The reference angle of internal friction is that obtained from a straight-line approximation of the Mohr-Coulomb failure envelope determined from Consolidated Undrained (CU) triaxial compression tests on undisturbed specimens.

For overconsolidated clays, K_{ooc} may be correlated with angle of internal friction and overconsolidation ratio, in accordance with Mayne and Kulhawy (1982). The plasticity index together with OCR may also be used for preliminary estimates of K_{ooc} as indicated by Brooker and Ireland (1965).

$$K_o = (1 - \sin\varphi')OCR^{\sin\varphi'} \qquad \text{(Mayne and Kulhawy, 1982)}$$

Overconsolidation Ratio - Clay

Overconsolidation ratio is defined as: OCR = σ'_p/σ'_{vo} where σ'_p is the pre-consolidation pressure considered to correspond with the maximum vertical effective stress to which the soil has been subjected, and σ'_{vo} is the current effective in situ vertical stress. The pre-consolidation pressure approximates a stress level where relatively small strains are separated from relatively large strains occurring on the virgin compression stress range. The reference OCR is usually based on laboratory oedometer tests carried out on undisturbed samples, and may thus be influenced by factors such as sample disturbance, strain rate effects and interpretation procedure.

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Various analytical and semi-empirical models for interpretation of pre-consolidation pressure from piezo-cone test data are available. Sandven (1990) presents a summary. The procedures are mostly "experimental" and as yet uncommon in practice. Chen and Mayne (1996) presented a direct correlation between net cone resistance and overconsolidation ratio for 205 clay sites around the world, as follows:

$$OCR = 0.317\ Q_t \qquad \text{(Chen and Mayne, 1996)}$$

The overconsolidation ratio may also be inferred from a geological assessment and from undrained strength ratios.

Geological factors concerning overconsolidation have been discussed under "in situ stress conditions - sand". An empirical procedure for estimation of OCR based on undrained strength ratio s_u/σ'_{vo} is given by Wroth (1984). The procedure uses the strength rebound parameter Λ. Guidance for selection of Λ and normally consolidated undrained strength ratio is given by Mayne (1988). Historically, much use has also been made of the Skempton (1957) relationship between normally consolidated undrained strength ratio and plasticity index I_p. This equation is useful for preliminary estimates, considering that I_p probably relates to φ' in some complex manner.

Undrained Shear Strength - Clay

No single undrained shear strength exists. The in situ undrained shear strength s_u depends on factors such as mode of failure, stress history, anisotropy, strain rate and temperature.

Various theoretical and empirical procedures are available to correlate q_c with s_u. Theoretical approaches use bearing capacity, cavity expansion or steady penetration solutions, all of which require a number of simplifying assumptions. Empirical approaches are more common in engineering practice because of difficulties in realistic soil modelling. An empirical correlation for soft to stiff, intact and relatively homogeneous clays is given by Battaglio et al. (1986) as follows:

$$s_u = (q_c - \sigma_{vo})/N_c$$

where s_u, σ_{vo} and q_c are in kPa. N_c is an empirical factor that ranges between 10 and 25, with the higher N_c factors applying to clays with a relatively low plasticity index, and vice versa. The reference undrained shear strength is that determined from in situ vane test results. The term σ_{vo} (total in situ vertical stress) becomes insignificant for stiff clays at shallow depth so that the equation reduces to $s_u = q_c/N_c$.

For specific design situations, a different s_u reference strength should be used. For example, offshore axial pile capacity predictions in accordance with API (2011) recommend s_u to be based on undrained triaxial compression tests, which are likely to yield lower s_u values than in situ vane tests. A site-specific or regional approach should generally be preferred. For example, N_c factors of 15 to 20 have been commonly used for firm to hard North Sea clays. They give reasonable strength estimates for s_u values determined from pocket penetrometer, torvane and Unconsolidated Undrained triaxial tests (UU) on Shelby tube samples obtained by hammer sampling and push sampling techniques. Lower N_c factors are generally appropriate for soft clays and higher factors for heavily overconsolidated clays.

If piezo-cone test data are available, then improved correlations are feasible because of the pore pressure information. Empirical correlations of piezo-cone test results with laboratory undrained shear strengths are commonly expressed, as follows:

$$s_u = q_n/N_k$$

N_k ranges typically between 8 and 30 with the higher N_k factors applying to heavily overconsolidated clays.

GTRC (2014) accounts for N_k variation according to B_q:

$$N_k = 10.5 - 4.6 \bullet \ln(B_q + 0.1)$$

where $B_q > -0.1$. The equation is based on 407 paired CPT and laboratory test results, particularly anisotropically consolidated triaxial compressive strength. Factoring of N_k can be applied by multiplying the calculated N_k factor by, for example, 0.85 and 1.2

Mayne et al. (2015) recommend a mean $N_k = 12$ with a standard deviation of 2.8 for correlation with laboratory anisotropically consolidated triaxial compressive strength. The recommendations are based on a

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study of 51 onshore and offshore clays and apply to normally consolidated to slightly overconsolidated clays with q_n values of typically less than 8 MPa. Slightly higher N_k values can be expected for average laboratory undrained shear strength, defined as the average of laboratory triaxial compression, simple shear and triaxial extension.

Clay Sensitivity

The sensitivity of a clay (S_t) is the ratio of undisturbed undrained shear strength to remoulded undrained shear strength. Sensitivity may be assessed from the CPT friction ratio R_f, in accordance with Schmertmann (1978):

$$S_t = N_s/R_f$$

where N_s is a correlation factor typically ranging between 5 and 10. The correlation is expected to be inaccurate for sensitive clays where uncertainty in very low values for sleeve friction may dominate results.

The reference S_t value is often taken to be that determined from undisturbed and remoulded laboratory unconsolidated undrained triaxial tests. This reference S_t value may differ from that determined from other tests, for example laboratory miniature vane tests. This is partly related to the definition of sensitivity. For vane tests, several measurements of undrained shear strength are possible:
- Intact (I) = undisturbed undrained shear strength as measured on an intact/undisturbed specimen.
- Intact-Residual (I-R) = measured post peak during initial shearing of the intact specimen.
- Intact-Vane Remoulded (I-VR) = measured after multiple-quick rotations of the vane after completion of the intact test.
- Hand Remoulded (HR) = steady state (post-peak if exists) resistance of hand remoulded test specimen.
- Hand Remoulded – Vane Remoulded (HR-VR) = steady state resistance of hand remoulded specimen measured after applying multiple-quick vane rotations.

Skempton and Northey (1952) present a correlation of sensitivity and laboratory liquidity index I_L. This correlation may allow a check on CPT-based interpretation of sensitivity.

Effective Shear Strength Parameters - Clay

Measurement of pore water pressures during penetration testing has led to development of interpretation procedures for estimation of effective stress parameters of cohesive soils. Background information may be found in Sandven (1990). Currently available procedures are evaluated to be "experimental" and are as yet not commonly adopted.

In general, CPT interpretation of effective shear strength parameters for clay and silt relies on soil behaviour-type classification.

It is noted that significant silt and sand fractions in a clay deposit will increase φ', while a significant clay fraction in silt will decrease φ'.

Masood and Mitchell (1993) provide an equation for the determination of φ' by combining sleeve friction with the Rankine earth-pressure theory. The equation is based on the following assumptions:
- Unit adhesion between soil and sleeve is negligible.
- Friction angle between soil and sleeve = $\varphi'/3$.
- Lateral earth pressure coefficient during penetration is equal to the Rankine coefficient of lateral earth pressure under passive conditions.

$$\frac{f_s}{\sigma'_{vo}} = \tan^2(45^\circ + \frac{\varphi'}{2})\tan(\frac{\varphi'}{3}) \qquad \text{(Masood and Mitchell, 1993)}$$

Mayne (2001) proposed an approximation of the Masood and Mitchell equation, as follows:

$$\varphi' = 30.8\left[\log(\frac{f_s}{\sigma'_{vo}}) + 1.26\right] \qquad \text{(Mayne, 2001)}$$

Mayne (2001) also proposed the following approximation of friction angle φ' based on pore pressure ratio B_q

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and the cone resistance number N_m (Senneset, Sandven and Janbu, 1989):

$$\varphi' = 29.5 B_q^{0.121}(0.256 + 0.336 B_q + \log N_m) \qquad \text{(Mayne, 2001)}$$

where

$$N_m = \frac{q_t - \sigma_{vo}}{\sigma'_{vo} + a}$$

where the cone resistance number N_m is dimensionless, total cone resistance q_t, total in situ vertical stress σ_{vo} and effective in situ vertical stress σ'_{vo} are in kPa.

Senneset et al. (1989) use the attraction value [a] as a function of soil type. In general the attraction value ranges from 5 to > 50 for both sands and clays and may be estimated directly from CPT results. The correlation is valid if the angle of plastification β is zero. In general a plastification angle of zero applies to medium sands and silts, sensitive clays and highly compressible clays.

Compressibility – Clay

Correlations between CPT data and compressibility parameters are viewed as indicative only, as discussed for sand compressibility.

The use of elasticity theory is common for analysis of undrained soil deformation behaviour. The adopted procedure is as follows:
(a) Estimation of undrained shear strength s_u from CPT data, as outlined above.
(b) Estimation of secant Young's moduli for undrained stress change E_u in general accordance with correlations based on s_u, as presented by Ladd et al. (1977).

Laboratory undrained triaxial tests carried out on undisturbed clay specimen form the basis for the E_u versus s_u correlations. Typical E_u/s_u ratios at a shear stress ratio of 0.3 range between about 300 and 900 for normally consolidated clays and E_u/s_u = 100 to 300 for heavily overconsolidated clay. Higher E_u/s_u ratios would apply to lower shear stress ratios, and vice versa.

Mitchell and Gardner (1976) present an approximate correlation of cone resistance with constrained modulus M (or coefficient of volume compressibility m_v, where $M = 1/m_v$). Typical ratios of M/q_c range between 1 and 8 for silts and clays. Refinements include q_c ranges and soil type (silt, clay, low plasticity, high plasticity, etc.). The correlation relies on the results of conventional laboratory oedometer tests carried out on undisturbed clay and silt samples. The constrained modulus can also be related (approximately) to secant Young's modulus E' and shear modulus G'.

It is noted that laboratory soil stiffness may differ from in situ stiffness because of inevitable sampling disturbance (in particular soil structure disturbance). In general, this implies that laboratory stiffness will usually be less than in situ stiffness.

Constrained Modulus M

Kulhawy and Mayne (1990) correlated constrained modulus M in clays with net cone resistance data. This relationship is based on data from 12 different test sites, with constrained moduli up to 60 MPa. The published standard deviation is 6.7 MPa.

$$M = 8.25\, q_n \qquad \text{(Kulhawy and Mayne, 1990)}$$

Shear Wave Velocity v_s – Clay

Hegazy and Mayne (2006) and Roberson and Cabal (2010) present empirical correlations between shear wave velocity and CPT parameters for a wide range of soils including clays, as described in “Shear Wave Velocity v_s – Sand” above. The Hegazy and Mayne correlation is sensitive to use of q_c or q_t. It should be used with caution for soils showing undrained or partially drained CPT response.

Mayne and Rix (1995) derived a correlation between shear wave velocity v_s and cone resistance q_c for intact and fissured clays. A database from Mayne and Rix (1993) was used including 31 different clay sites.

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$$v_s = 1.75 q_c^{0.627} \qquad \text{(Mayne and Rix, 1995)}$$

where shear wave velocity v_s is in m/s and cone resistance q_c is in kPa.

Shear Modulus G_{max}

Mayne and Rix (1993) determined a relationship between G_{max} and q_c by studying 481 data sets from 31 sites all over the world. G_{max} ranged between about 0.7 MPa and 800 MPa.

$$G_{max} = 2.78\, q_c^{1.335} \qquad \text{(Mayne and Rix, 1993)}$$

where G_{max} and q_c are in kPa.

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INTRODUCTION

This document describes survey of horizontal and elevation/depth reference points for geotechnical and/or environmental data acquisition in a marine environment.

National and international standards for geotechnical and/or environmental data acquisition (as ASTM, BSI, CEN and ISO) require such surveys, but do not describe procedural details. This document summarises common practice.

PROCEDURE

The procedure for positioning survey and depth measurement depends on the agreed project specifications. For example, water level correction and subsurface positioning may not be part of the activities agreed upon. Some or all of the following steps can apply:
- definition of the type of survey and the target location;
- set-up and initial checks of the survey system and depth measurement system;
- surface positioning survey of the reference point, i.e. the determination of grid coordinates;
- subsurface positioning survey, i.e. adjustment of the surface positioning results for underwater offset;
- measurement of the water depth;
- calculation of elevation of seafloor or a data point for the seabed relative to a vertical datum, e.g. water level correction.

This document uses the terms seafloor and seabed. Seafloor is the underwater ground surface, i.e. the plane separating water and ground (soil, rock, made ground). The seabed is the ground below seafloor.

SURVEY CLASSIFICATIONS

Positioning surveys require specific systems and procedures, such as those summarised below for marine applications. The International Hydrographic Organization (IHO, 2008) defines four orders of hydrographic survey (Table 1). The term “depth” refers here to water depth, i.e. the vertical distance between water level and seafloor. Water level can be expressed with reference to, for example, Lowest Astronomical Tide LAT.

Table 1 – Summary of IHO Classification

IHO Order	Special	1a	1b	2
Description of Areas	Areas where under-keel clearance is critical	Areas shallower than 100 m where under-keel clearance is less critical but features of concern to surface shipping may exist	Areas shallower than 100 m where under-keel clearance is not considered to be an issue for the type of surface shipping expected to transit the area	Areas generally deeper than 100 m where a general description of the seafloor is considered adequate
Maximum Allowable Total Horizontal Uncertainty 95 % Confidence Level	2 m	5 m + 5 % of depth	5 m + 5 % of depth	20 m + 10 % of depth
Maximum Allowable Total Vertical Uncertainty 95 % Confidence Level	a = 0.25 m b = 0.0075	a = 0.5 m b = 0.013	a = 0.5 m b = 0.013	a = 1.0 m b = 0.023
Full Seafloor Search	Required	Required	Not required	Not required
Feature Detection	Cubic features > 1 m	Cubic features > 2 m in depths up to 40 m; 10 % of depth beyond 40 m	Not applicable	Not applicable
Recommended Maximum Line Spacing	Not defined as full seafloor search is required	Not defined as full seafloor search is required	3 x average depth or 25 m, whichever is greater	4 x average depth

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Note: The use of coefficients a and b is as follows:

$$\pm\sqrt{[a^2 + (b * d)^2]}$$

where:

a represents that portion of the uncertainty that does not vary with water depth
b is a coefficient which represents that portion of the uncertainty that varies with water depth
d is the water depth
b*d represents that portion of the uncertainty that varies with water depth.

Figure 1 illustrates the effect of coefficients a and b.

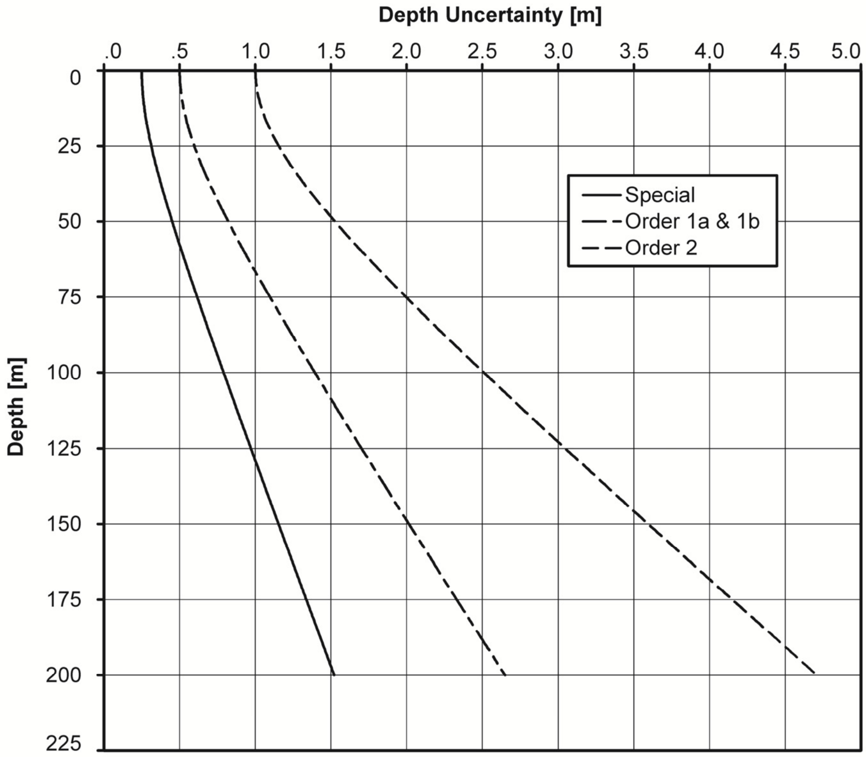

Figure 1 IHO water depth uncertainty

IHO Survey Classification - Offshore Practice Examples

The IHO Special Order Survey is exceptional in geotechnical and/or environmental data acquisition. A Special Order system set-up may be comprised of: RTK DGPS; a multibeam echo sounder; a motion compensator, and a conductivity temperature depth (CTD) probe. Subsurface positioning is uncommon in limited water depths.

An IHO Order 1a and 1b survey system set-up may include: high-accuracy DGPS; long baseline (LBL) subsurface positioning; a CTD probe with Digiquartz pressure sensor; a barometer; and a tide gauge.

IHO Order 2 surveys are common in geotechnical and/or environmental data acquisition. Such system set-ups could include: DGPS; ultra short baseline (USBL) subsurface positioning (IMCA, 2011); CTD probe; single beam echo sounder or direct sounding by drill pipe; a motion compensator; and predicted tide correction.

These are examples of the simplest set-ups. Independent measurements are often made using a redundant system (OGP, 2011). For example, surface position may be determined by two independent DGPS systems or direct sounding by drill pipe and echo sounding.

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Comments on Uncertainty Budget

IHO Order and offshore system set-ups involve relatively complex uncertainty budgets (uncertainty estimates). IHO considers total propagated uncertainties for the reference point on the seafloor. For example, horizontal positioning must not only consider the uncertainty of a DGPS antenna position, but also uncertainty in offset between antenna and actual position of a tool on the seafloor.

Horizontal positioning

- DGPS - antenna position uncertainty typically in the order of 1 to 2 metres.
- High accuracy DGPS - antenna position uncertainty typically in the order of 0.2 m.
- RTK DGPS – antenna position uncertainty typically in the order of centimetres.
- Gyro compass – uncertainty typically in the order of 0.5^{o} to 1^{o}.

DGPS uncertainty contributions include the geodetic network, vessel dynamics and antenna offset. Continuous logging on location allows some quantification of position uncertainty.

Subsurface positioning

- LBL system: receiver position uncertainty typically in the order of 1 metre.
- USBL system: uncertainty of typically 0.5 m plus 1 % of distance between transducer and transceiver.

Uncertainty contributions include timing, ray bending, sound absorption, noise and offset.

Water depth measurement

- Direct sounding by drill pipe: uncertainty of typically about 1 m plus 0.5 % of measured mean water depth.
- Echo sounder: uncertainty of typically about 0.3 m plus 1 % of measured mean water depth.
- Digiquartz probe: probe position uncertainty of typically about 0.2 m plus 0.1 % of measured mean water depth.
- Motion compensator: heave measurements have a typical uncertainty of 0.05 m, and roll and pitch an uncertainty of about 0.1°, relative to the mounting of the unit itself.

The pressure sensor estimates are corrected for atmospheric pressure. The echo sounder estimate typically incorporates CTD sound velocity checks, motion compensation, and transducer draught, including vessel squat correction. Vessel squat is a vertical displacement of the hull as a vessel moves, and is determined by water depth and the vessel shape and size. The direct sounding estimate includes uncertainties related to tape measurement, heave, drill pipe length variation due to self-weight and temperature change, drill pipe bending and offset from vertical axis.

Tide correction

- Predicted tides: correction uncertainty typically in the order of 0.2 m to 1 m, depending on tidal range and meteorological circumstances.
- High accuracy DGPS: antenna position uncertainty typically in the order of 0.3 m.
- Tide gauge: correction uncertainty typically in the order of 0.1 m.
- RTK DGPS: antenna position uncertainty typically in the order of 0.1 m.

Uncertainty budgets can be project-specific. Soft soils, for example, can introduce uncertainty in underwater vertical position of measurement. A water pressure measurement tool mounted on an underwater frame may sink into the soil, thus affecting the measurement. Insufficient acoustic contrast between water and soft soil may affect echo sounder water depth measurements.

An irregular or sloping seafloor may affect echo sounder measurements. An echo sounder determines the earliest arrival of acoustic waves within the beam area. The highest points within the beam are assumed to correlate with the seafloor position, and thus yield the "water depth".

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DEPTH BELOW SEAFLOOR

This section describes estimation of depth below seafloor of a data point or measurement point acquired by borehole logging, in situ testing, and physical sampling and laboratory testing.

ISO (2014) provides depth accuracy classes, as shown in Table 2. Class Z4 applies as default, except for samplers with no fixed seafloor reference, where Z5 applies as default. Note that ISO (2014) uses accuracy class and application class interchangeably. A definition is given for application class and not for accuracy class. Application classes are defined in terms of “classification of equipment based on achievable level of accuracy”. This is interpreted to mean achievable under favourable conditions.

Table 2 – Depth Accuracy Classes for Data Point Measurements Relative to Seafloor

Depth Accuracy Class	Maximum Data Point Depth Uncertainty [m]
Z1	0.1
Z2	0.5
Z3	1.0
Z4	2.0
Z5	> 2.0

ISO (2014) includes guidance on factors to consider for data point depth uncertainty. One of the factors is the position of a sample or test specimen with a sampler.

Peuchen et al. (2005) present the following expression for depth uncertainty assessment for in situ testing, i.e. excluding considerations for sampling and laboratory testing:

$$\Delta z = \pm\sqrt{[a^2 + (b * d)^2 + (c * z)^2]}$$

where:

- a constant depth uncertainty, i.e. the sum of all uncertainties that do not vary with depth below seafloor in metres
- b uncertainty dependent on water depth, i.e. the sum of all uncertainties that are water depth dependent
- c uncertainty dependent on data point depth below seafloor, i.e. the sum of all uncertainties that are data point depth dependent
- d water depth in metres
- z data point depth in metres below to seafloor
- Δz data point depth uncertainty in metres (95 % confidence level)

Tables 3 to 5 present coefficients and accompanying premises.

Table 3 - Coefficients for Data Point Uncertainty Assessment – In Situ Testing

Deployment System	Data Point Depth Uncertainty Δz		
	A	b	c
Vessel drilling – favourable	0.4 m	0.003	0.003
Vessel drilling – adverse	1.0 m	0.005	0.004
Non-drilling – favourable	0.2 m	0	0.01
Non-drilling – adverse	0.8 m	0	0.02
Note: resolution estimated at 50 % of uncertainty			

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Table 4 - Premise to Estimated Data Point Depth Uncertainty – In Situ Testing and Vessel Drilling Deployment

Characteristics	Marine Setting	
	Favourable	Adverse
Vessel - horizontal position	Variation within 5 m of target	Variation within 5 m of target
Vessel heave	1 m at “hook” point	3 m at “hook” point
Tidal variation	1.5 m, with correction for tidal variation by pressure sensor mounted on seabed frame	3 m, with correction for tidal variation by pressure sensor mounted on seabed frame
Seafloor	Firm and level	Very soft seabed soils or very rugged seafloor
Drill pipe checkpoint	Touchdown on seabed frame at borehole start	Touchdown on seabed frame at borehole start
Drill pipe bending	None	Minor
Borehole orientation	Vertical	Inclined at average 2° from vertical from sea level to test depth z

Table 5 - Premise to Estimated Data Point Depth Uncertainty – In Situ Testing and Non-Drilling Deployment

Characteristics	Marine Setting	
	Favourable	Adverse
Vessel - horizontal position	Variation within 5 m of target	Variation within 5 m of target
Vessel heave	1 m at “hook” point	3 m at “hook” point
Tidal variation	1.5 m	3 m
Seafloor	Firm and level	Very soft seabed soils or very rugged seafloor
Orientation of Penetration	Vertical at start, with correction for measured inclination	Inclined at average 5° from vertical from seafloor to test depth z

Definition of seafloor is difficult for extremely soft ground. Reaction equipment may penetrate unnoticed into a near-fluid zone of the seabed. Settlement may also continue during testing (Bouwmeester et al., 2009).

Seabed frame settlement is likely to be governed by the following factors:

(1) Descent velocity and penetration into seabed, including possible erosion (scouring) caused by seabed frame descent and resulting water overpressures.
(2) Non-centric loading during touchdown and testing.
(3) Variable on-bottom weight of reaction equipment, because of drilling, sampling and testing activities and because of tensioning and hysteresis forces in a heave compensation system.
(4) Consolidation of seabed sediments.

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Symbol	Unit	Quantity
I - GENERAL		
L	m	Length
B	m	Width
D	m	Diameter
d	m	Depth
h	m	Height or thickness
z	m	Penetration or depth below reference level (usually ground surface)
A	m^2	Area
V	m^3	Volume
W	kN	Weight
t	s	Time
v	m/s	Velocity
a	m/s^2	Acceleration
g	m/s^2	Acceleration due to gravity ($g = 9.81\ m/s^2$)
m	kg	Mass
ρ	kg/m^3	Density
π	-	Mathematical constant (= 3.14159)
e	-	Base of natural logarithm (= 2.71828)
ln	-	Natural logarithm
log	-	Logarithm base 10
II - STRESS AND STRAIN		
P_a	kPa	Atmospheric pressure
u	MPa	Pore water pressure
u_o	MPa	Hydrostatic pore pressure relative to seafloor or phreatic surface
σ	kPa	Total stress
σ'	kPa	Effective stress
τ	kPa	Shear stress
$\sigma_1,\sigma_2,\sigma_3$	kPa	Principal stresses
σ'_{ho}	kPa	Effective in situ horizontal stress
σ_{vo}	kPa	Total in situ vertical stress relative to ground surface or phreatic surface
σ'_{vo}	kPa	Effective in situ vertical stress (or p'_o)
σ'_h	kPa	Effective horizontal stress
σ'_v	kPa	Effective vertical stress
r_u	-	Pore pressure ratio [$= u/\sigma_{vo}$]
p'	kPa	Mean effective stress [$= (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$]
q	kPa	Principal deviator stress [$= \sigma'_1 - \sigma'_3$] or [$= \sigma_1 - \sigma_3$]
s'	kPa	Mean effective stress in s'-t space [$= (\sigma'_1 + \sigma'_3)/2$]
t	kPa	Shear stress in s'-t space [$= (\sigma'_1 - \sigma'_3)/2$] or [$= (\sigma_1 - \sigma_3)/2$]
ε	-	Linear strain
$\varepsilon_1,\varepsilon_2,\varepsilon_3$	-	Principal strains
ε_v	-	Volumetric strain
γ	-	Shear strain
ν	-	Poisson's ratio
ν_u	-	Poisson's ratio for undrained stress change
ν_d	-	Poisson's ratio for drained stress change
E	MPa	Modulus of linear deformation (Young's modulus)
E_u	MPa	Modulus of linear deformation (Young's modulus for undrained stress change)
E_d	MPa	Modulus of linear deformation (Young's modulus for drained stress change)
G	MPa	Modulus of shear deformation (shear modulus)
G_{max}	MPa	Shear modulus at small strain
I_r	-	Rigidity index [$= G/\tau_{max}$ or G/s_u]
K	MPa	Modulus of compressibility (bulk modulus)
M	MPa	Constrained modulus [$= 1/m_v$]
μ	-	Coefficient of friction
η	kPa.s	Coefficient of viscosity

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Symbol	Unit	Quantity
III - PHYSICAL CHARACTERISTICS OF GROUND		
(a) Density and Unit Weights		
γ	kN/m^3	Unit weight of ground (or bulk unit weight or total unit weight)
γ_d	kN/m^3	Unit weight of dry ground
γ_s	kN/m^3	Unit weight of solid particles
γ_w	kN/m^3	Unit weight of water
γ_{pf}	kN/m^3	Unit weight of pore fluid
γ_{dmin}	kN/m^3	Minimum index (dry) unit weight
γ_{dmax}	kN/m^3	Maximum index (dry) unit weight
γ' or γ_{sub}	kN/m^3	Unit weight of submerged ground
ρ	Mg/m^3 [= t/m^3]	Density of ground
ρ_d	Mg/m^3 [= t/m^3]	Density of dry ground
ρ_s	Mg/m^3 [= t/m^3]	Density of solid particles
ρ_w	Mg/m^3 [= t/m^3]	Density of water
D_r	-, %	Relative density [= $I_D = \gamma_{dmax} (\gamma_d-\gamma_{dmin})/\gamma_d(\gamma_{dmax}-\gamma_{dmin}) = (e_{max}-e)/(e_{max}-e_{min})$]
v	-	Specific volume [= 1+e]
e	-	Void ratio
e_0	-	Initial void ratio
e_0	-	Void ratio at σ'_{vo}
e_{max}	-	Maximum index void ratio
e_{min}	-	Minimum index void ratio
I_D	-, %	Density index [= D_r]
R_D	-, %	Dry density ratio [= γ_d/γ_{dmax}]
n	-, %	Porosity
w	%	Water content
S_r	%	Degree of saturation
r	-, g/kg	Salinity of pore fluid [= ratio of mass of salt to mass of pore fluid]
R	g/l	Salinity of fluid [= ratio of mass of salt to volume of distilled water]
s	g/l	Salinity of fluid [= ratio of mass of salt to volume of fluid]
S	g/kg	Salinity of seawater [= ratio of mass of salt to mass of seawater]
(b) Consistency		
w_L	%	Liquid limit
w_P	%	Plastic limit
I_P	%	Plasticity index [= w_L - w_P]
I_L	%	Liquidity index [= $(w - w_P)/(w_L - w_P)$]
I_C	%	Consistency index [= $(w_L - w)/(w_L - w_P)$]
A	-, %	Activity [= ratio of plasticity index to percentage by weight of clay-size particles]
(c) Particle Size		
D	mm	Particle diameter
D_n	mm	Particle diameter, where n% of the dry mass of ground has a smaller particle diameter
C_u	-	Uniformity coefficient [= D_{60}/D_{10}]
C_c	-	Curvature coefficient [= $(D_{30})^2/D_{10}D_{60}$]
(d) Acoustic and Dynamic Properties		
v_p	m/s	P-wave velocity (compression wave velocity)
v_s	m/s	S-wave velocity (shear wave velocity)
v_{s1}	m/s	S-wave velocity normalised to 100 kPa in situ vertical stress
D	-, %	Damping ratio of ground

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Symbol	Unit	Quantity
(e) Hydraulic Properties		
k	m/s	Coefficient of permeability
k_v	m/s	Coefficient of vertical permeability
k_h	m/s	Coefficient of horizontal permeability
i	-	Hydraulic gradient
(f) Thermal and Electrical Properties		
T	K, °C	Temperature
k	W/(m·K)	Thermal conductivity
a_L	1/°C	Thermal expansion coefficient (linear)
α	m^2/s	Thermal diffusion coefficient
ρ	Ω.m	Electrical resistivity
K	S/m	Electrical conductivity
(g) Magnetic Properties		
B	T	Magnetic flux density (or magnetic induction)
(h) Radioactive Properties		
γ	CPS	Natural gamma ray

IV - MECHANICAL CHARACTERISTICS OF GROUND

Symbol	Unit	Quantity
(a) Cone Penetration Test (CPT)		
q_c	MPa	Cone resistance
q_{c1}	MPa	Cone resistance normalised to 100 kPa effective in situ vertical stress
f_s	MPa	Sleeve friction
f_t	MPa	Sleeve friction corrected for pore pressures acting on the end areas of the friction sleeve
R_f	%	Ratio of sleeve friction to cone resistance
R_{ft}	%	Ratio of sleeve friction to corrected cone resistance (f_s/q_t or f_t/q_t)
u_1	MPa	Pore pressure at the face of the cone
u_2	MPa	Pore pressure at the cylindrical extension above the base of the cone or in the gap between the friction sleeve and the cone
u_2*	MPa	Pore pressure u_2, but derived rather than measured
u_3	MPa	Pore pressure immediately above the friction sleeve or in the gap above the friction sleeve
K	-	Adjustment factor for ratio of pore pressure at u_1 to u_2 location
q_n	MPa	Net cone resistance
q_t	MPa	Corrected cone resistance (or total cone resistance)
B_q	-	Pore pressure ratio
Q_t	-	Normalized cone resistance [= q_n/σ'_{vo}]
Q_{tn}	-	Normalized cone resistance with variable stress exponent
F_r	%	Normalized friction ratio [= f_t/q_n]
N_c	-	Cone factor between q_c and s_u
N_k	-	Cone factor between q_n and s_u
I_c	-	Soil behaviour type index (for Q_{tn} and F_r)
I_{SBT}	-	Soil behaviour type index (for q_c and R_f)
(b) Standard Penetration Test (SPT)		
N	Blows/0.3 m	SPT blow count
N_{60}	Blows/0.3 m	SPT blow count normalised to 60 % energy
$N_{1,60}$	Blows/0.3 m	SPT blow count normalised to 60 % energy and to 100 kPa effective in situ vertical stress

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Symbol	Unit	Quantity

(c) Strength of Soil

s_u	kPa	Undrained shear strength (or c_u)
s_u/σ'_{vo}	-	Undrained strength ratio
κ	kPa/m	Rate of increase of undrained shear strength with depth (linear)
c'	kPa	Effective cohesion intercept
φ'	°(deg)	Effective angle of internal friction
φ'_{cv}	°(deg)	Effective angle of internal friction at large strain
ε_{50}	%	Strain at 50 % of peak deviator stress (or ε_c)
E_{50}	MPa	Young's modulus at 50 % of peak deviator stress
$s_{u;r}$	kPa	Undrained shear strength of remoulded soil
$s_{u;ar}$	kPa	Undrained shear strength of aged remoulded soil
s_R	kPa	Undrained residual shear strength
S_t	-	Sensitivity [= $s_u/s_{u;r}$ or s_u/s_R]
T_x	-	Thixotropy strength ratio [$T_x(t) = s_{u;ar}(t)/s_{u;r}$]
σ'_c	kPa	Effective consolidation pressure
M	-	Gradient of critical state line when projected onto a constant volume plane
A	-	Pore pressure coefficient for anisotropic pressure increment
B	-	Pore pressure coefficient for isotropic pressure increment

(d) Strength of Rock

$I_{s(50)}$	MPa	Point load strength index
σ_c	MPa	Uni-axial compressive strength

(e) Consolidation (One Dimensional)

σ'_p	kPa	Effective preconsolidation pressure (or effective vertical yield stress in situ)
σ^*_{ve}	kPa	Effective vertical stress on ICL at e_0
σ'_{vy}	kPa	Effective vertical yield stress in situ (or effective preconsolidation pressure)
C_c	-	Compression index
C^*_c	-	Intrinsic compression index [= $e^*_{100} - e^*_{1000}$]
C_s	-	Swelling index (or re-compression)
CR	-	Primary compression ratio [= $C_c/(1+e_0)$]
RR	-	Recompression ratio [= $C_s/(1+e_0)$]
e_L	-	Void ratio at liquid limit w_L
e^*_{100}	-	Void ratio at σ'_v = 100 kPa during one-dimensional intrinsic compression
e^*_{1000}	-	Void ratio at σ'_v = 1000 kPa during one-dimensional intrinsic compression
C_α	-	Coefficient of secondary compression (primary compression)
$C_{\alpha s}$	-	Coefficient of secondary compression (swelling/re-compression)
c_v	m^2/s	Coefficient of consolidation
H	m	Drainage path length
ICL	-	Intrinsic compression line (Burland 1990)
I_v	-	Void index [= $(e_0 - e^*_{100})/C^*_c$]
m_v	m^2/MN	Coefficient of volume compressibility
M	MPa	Constrained modulus [= $1/m_v$]
p	kPa	Vertical pressure
OCR	-	Overconsolidation ratio [= σ'_p/σ'_{vo}]
SCC	-	Sedimentation compression curve
SCL	-	Sedimentation compression line (Burland 1990)
S_σ	-	Stress sensitivity [= $\sigma'_{vy}/\sigma^*_{ve}$]
YSR	-	Yield stress ratio [= $\sigma'_{vy}/\sigma'_{vo}$]

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V - GEOTECHNICAL DESIGN

(a) Partial Factors

γ_d	-	Factor related to model uncertainty or other circumstances
γ_f	-	Partial action factor (load factor)
γ_m	-	Partial material factor (partial safety factor)
γ_R	-	Partial resistance factor (partial safety factor)

(b) Seismicity

a_g	m/s^2	Effective peak ground acceleration (design ground acceleration)
d_g	m	Peak ground displacement
α	-	Acceleration ratio [= a_g/g]
τ_c	kPa	Seismic shear stress

(c) Compaction

ρ_{dmax}	Mg/m^3 [= t/m^3]	Maximum dry density
ρ_{max}	Mg/m^3 [= t/m^3]	Maximum density
w_{opt}	%	Optimum moisture content

(d) Earth Pressure

δ	°(deg)	Angle of interface friction (between ground and foundation)
K	-	Coefficient of lateral earth pressure
K_a	-	Coefficient of active earth pressure
K_{ac}	-	Coefficient of active earth pressure for total stress analysis
K_p	-	Coefficient of passive earth pressure
K_{pc}	-	Coefficient of passive earth pressure for total stress analysis
K_o	-	Coefficient of earth pressure at rest
K_{onc}	-	K_o for normally consolidated soil
K_{ooc}	-	K_o for overconsolidated soil

(e) Foundations

A	m^2	Total foundation area
A'	m^2	Effective foundation area
B'	m	Effective width of foundation
E_s	MN/m^3	Modulus of subgrade reaction
k	MPa/m	Rate of change of modulus of subgrade reaction E_s with depth z
L'	m	Effective length of foundation
H	MN	Horizontal external force or action
V	MN	Vertical external force or action
M	MN.m	External moment
T	MN.m	External torsion moment
Q	MN	Total vertical resistance of a foundation/pile
Q_p	MN	End bearing of pile
Q_s	MN	Shaft resistance of pile
q_p	MPa	Unit end bearing
q_{lim}	MPa	Limit unit end bearing
f	kPa	Unit skin friction (or q_s)
f_{lim}	kPa	Limit unit skin friction
p	MN/m	Lateral resistance per unit length of pile
p_{lim}	MN/m	Limit lateral resistance per unit length of pile
s	m	Settlement
t	MN/m	Skin friction per unit length of pile
y	mm	Lateral pile deflection
z	mm	Axial pile displacement
α	-	Adhesion factor between ground and foundation (= f/s_u)
β	-	Adhesion factor between ground and foundation (= f/σ'_v or f/σ'_{vo})

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Symbol	Unit	Quantity
δ	°(deg)	Angle of interface friction (between ground and foundation)
δ_{cv}	°(deg)	Constant volume or critical-state angle of interface friction (between ground and foundation)
N_c, N_q, N_γ	-	Bearing capacity factors
K_c, K_q, K_γ	-	Bearing capacity correction factors for inclined forces or actions, foundation shape and depth of embedment
i_c, i_q, i_γ	-	Bearing capacity correction factors for external force inclined from vertical shape
s_c, s_q, s_γ	-	Bearing capacity correction factors for foundation shape
d_c, d_q, d_γ	-	Bearing capacity correction factors for foundation embedment

Signs:
- A "prime" applies to effective stress.
- A "bar" above a symbol relates to average properties.
- A "dot" above a symbol denotes derivative with respect to time.
- The prefix "Δ" denotes an increment or a change.
- A “star” after a symbol denotes value corrected for pore fluid salinity.

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